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TRANSACTIONS.

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544.

(Vol. XXVII.—August, 1892.)

DESCRIPTION OF THE IRON COAL-PIER, NORFOLK AND WESTERN RAILROAD COMPANY, AT LAMBERT'S POINT, NORFOLK, VA., AND SOME OF THE METHODS USED IN ITS CONSTRUCTION.

By W. W. COE, M. Am. Soc. C. E.

WITH DISCUSSION.*

The river bottom at Lambert's Point is particularly well adapted to timber pile work and, incidentally, to iron piling. In the vicinity of the wooden coal-pier, the top stratum is a fine white sand varying in thickness from 1 to 3 feet. It has been packed hard by action of the water, and offers considerable resistance to the penetration of piles, but is of considerable use in increasing their lateral stability. Underlying this sand we find a thick stratum of sandy marl through which the penetration of piles is comparatively easy, a pile 7 inches in diameter at the point being driven about 18 inches under a blow of a 3 000-pound hammer falling 15 feet. The depth of this marl is slightly irregular, but will average about 40 feet. Directly under this there is a stratum of sand on which all piles bring up well, and through which, in our construction at Lambert's Point, they have never been driven. It may be stated here, however, that from an artesian well-boring near

* See page 129.

by, it has been noted that the marl with an occasional stratum of sand extends to a depth of 1 200 feet, where calcareous rock is found.

In locating the iron pier, it was considered advisable to make use as much as possible of the approach track leading to the old coal-pier, there being no doubt of the ability to supply both piers with coal from a short single track. If at any time a separate approach should be decided upon, there will be no difficulty in its construction. It was also decided to build the new approach trestle, connecting the iron pier with the old approach, of timber, as in the event of rebuilding the old pier in iron the same design could be used for both. The iron pier proper extends from the bulkhead a distance of 805 feet to the Port Warden's Line, and in general design is similar to the old pier, which during six years of continual service has given entire satisfaction. There are two elevated delivery tracks for loaded coal cars, one on each side of the pier, built on a grade of 9 inches per 100 feet descending from the summit, which is on the approach to the river end of the pier. The return track for the empty cars is situated between the delivery tracks, with a grade of 2 feet 6 inches per 100 feet, descending from the summit before mentioned to the grade of the yard tracks. Connecting the delivery tracks with the return track are two shifting tables with an end movement of 8 feet overcoming a grade of 8 inches, which is the difference in height of the delivery and return tracks at the tables. When at rest they are in line with the delivery tracks and at the same grade. After receiving the empty cars, they move by gravity to the line and grade of the return track, and allow the cars to leave the tables without assistance. The tables, when thus relieved of the weight of the empty cars, are at once brought back by counter-balance weights to the delivery tracks.

By the system thus adopted, loaded cars, after being placed by an engine beyond the summit of the grade, are henceforth controlled by gravity entirely, and, when empty, are formed into a train at the foot of the grade. The foundations for the pier consist of hollow iron piles in bents 36 feet apart, four piles in each bent. These piles and in fact all of the iron-work, both sub-structure and superstructure, are protected from injury from contact with vessels by having a timber protection of creosoted pine poles which prevents a nearer approach than 8 feet. This timber protection was built first, and forms a pier alongside of which vessels land, and was found to be of great

value as a platform for carrying the necessary traveler and the pump used for sinking the piles and in erecting superstructure. The traveler was made of sufficient height and span to admit of its overreaching the superstructure, and it could thus travel the entire length of the pier at all times. It was composed of two Howe trusses, 50 feet long, supported by double posts on each side, thoroughly braced together and resting on large truck frames, which also carried on one side a 50-horse-power boiler and a Mundy hoisting engine, and on the other side, for working the jet, a Worthington duplex pump of 890 gallons' capacity per minute.

The iron piles were made of wrought-iron pipes, 12 inches in diameter, one-half inch thick, in sections of from 14 to 20 feet, with flush joints and with a cast-iron disc 4 feet in diameter at the base of each pile. In forming the piles, they were brought to the traveler in sections and put together, an iron rust joint connecting the discs with the piles. When ready for sinking, which would be after forty-eight hours, the time required for the iron rust to set, the piles composing each bent would be placed accurately in position.

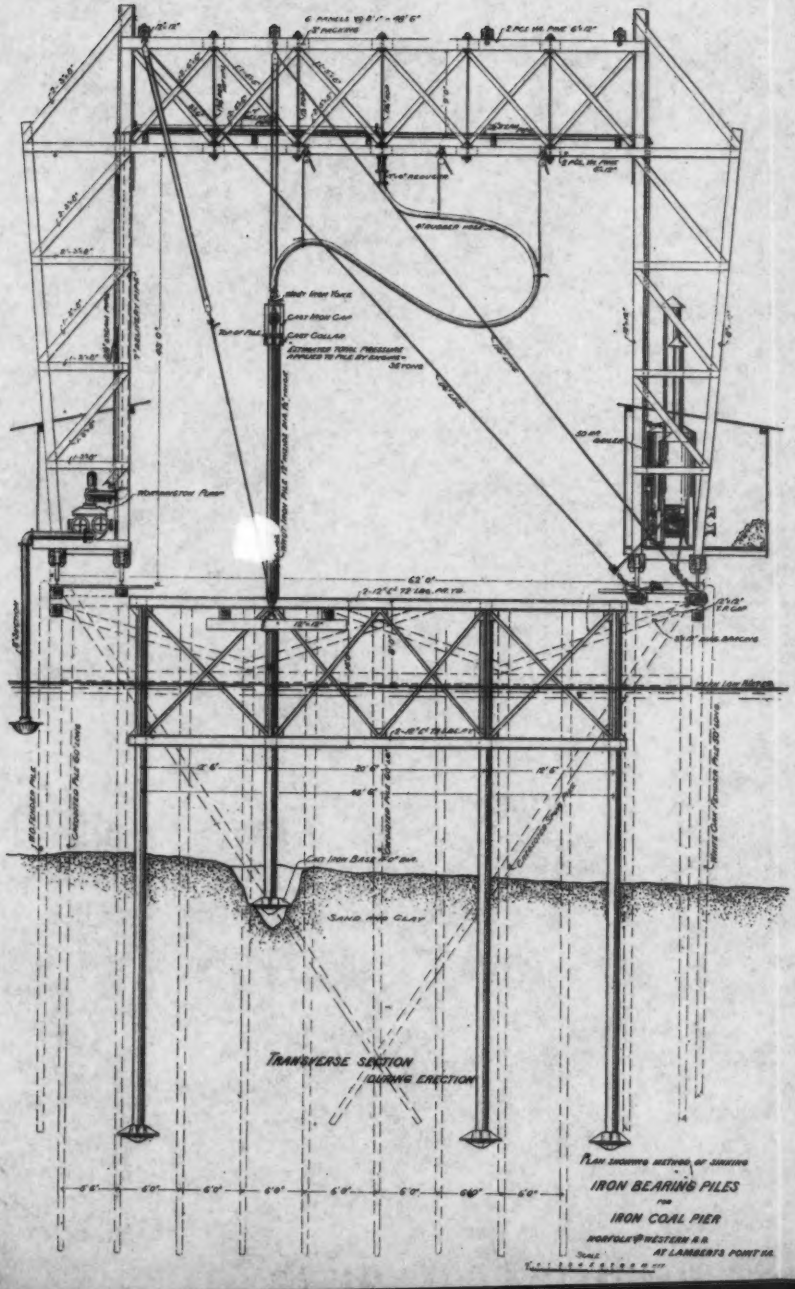
The transverse sub-bracing composed of two lines of 12-inch channels, 12 feet apart vertically, and connected with double angles forming a rigid frame, would then be placed one on each side of the bent. At proper distances on the inside of the channels, and well-bolted to them, are cast-iron clamps with semi-circular faces and lugs for accurately clamping the piles. Large bolts pass through both clamps and channels, and, when tightened up, form a very close connection between the piles and frame. These bolts would not be quite tightened up until after the piles had been driven down and cut off at the correct height, but the frame acted as a guide to the piles and kept them in position while they were going down. In sinking the piles, which varied in length from 45 to 57 feet, a top covering was placed on them and securely clamped by bolting, and a rubber hose connection 4 inches in diameter was made between the pump and pile to be sunk.

A water pressure of from 30 to 60 pounds was then put on, operating ten jets at the bottom of the disc. One of these $1\frac{3}{8}$ inches in diameter acted perpendicularly downward, and nine, $\frac{3}{8}$ inches in diameter, worked on radial lines with a declination of 45 degrees. The weight of the piles assisted by these jets would generally carry them down about 6 feet, when it was found necessary to apply blocks and tackle and to pull

down with an estimated force of 35 tons, using the hoisting engine for this purpose. By this means the piles were sunk to the lower part of the marl stratum, and it was found impossible to pull them down farther. It was then decided to apply a test weight to each of the piles, using for this purpose a timber platform balanced on the piles and loaded with pig iron, the total weight being 64 tons. Under this there was settlement in nearly every pile, varying from one-half inch to 6 feet in extreme cases, but averaging 13 inches for all. This settlement was undoubtedly due to the tearing up and softening of the bottom by the force of the water-jet. The weight remained on each pile for four hours after settlement had ceased, and the piles were then considered safe for any actual load that they might be called upon to sustain. It might be proper to remark here that the pier has been in constant use by heavy engines and 60 000 pounds' capacity coal cars for nearly a year, and that no appreciable settlement has occurred. After this, the piles were cut off at the regular height by a special pipe-cutting tool, and were then filled with concrete of broken brick, sand and Portland cement. The bolts in the sub-bracing were then tightened up and the substructure was complete, ready to receive the superstructure. The average weight of the piles is 4 800 pounds including the cast-iron discs, and the total weight of the substructure is 736 000 pounds. The superstructure consists of latticed channel columns, four in each bent, bearing directly on the iron pile heads, and secured by bolting to the channels of the sub-bracing.

Longitudinal girders for the delivery tracks bear directly on the heads of the columns except at each end of the pier, where, owing to the change of track, cross-girders were used. Longitudinal girders for the return track were carried by cross-girders, riveted to the inside of the columns. All the bracing for columns is of latticed angles and channels with riveted connections; the total weight of the iron superstructure is 1 826 000 pounds.

This article should not be closed without reference to the very careful and intelligent manner in which this pier was designed by the contractors, Messrs. Cofrode & Evans, in consultation with Mr. Walter L. Keen, engineer in charge of our improvements at Norfolk, and with the writer. Tribute should also be paid to the very careful as well as energetic manner in which the work was conducted by Mr. Keen and the contractors, the work being, to those connected with it at least, in a very large degree experimental in its nature.



PLAN SHOWING METHOD OF DRIVING

IRON BEARING PILES

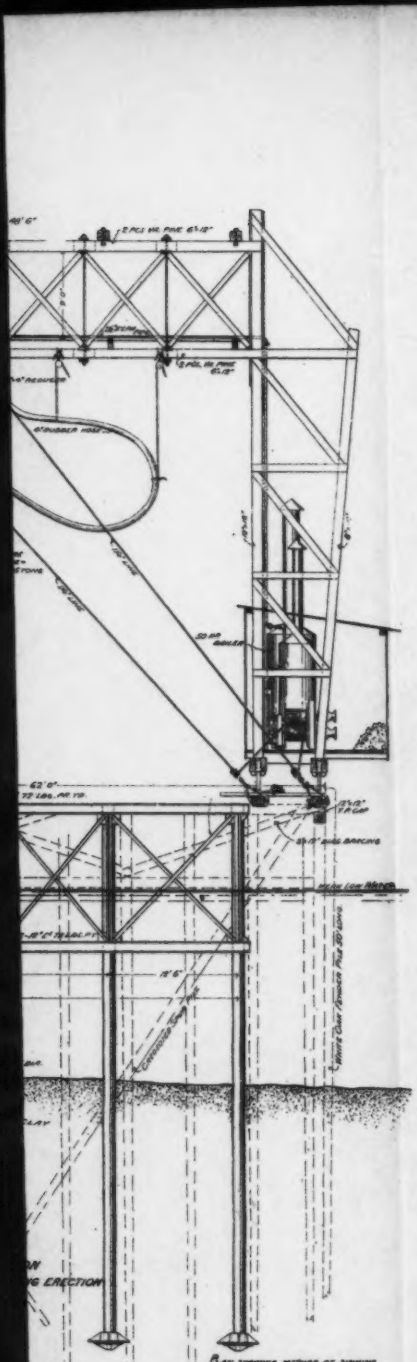
FOR

IRON COAL PIER

NORFOLK & WESTERN R.R.

AT LAMBERTS POINT VA.

SCALE



PLAN SHOWING METHOD OF DRIVING

IRON BEARING PILES

FOR

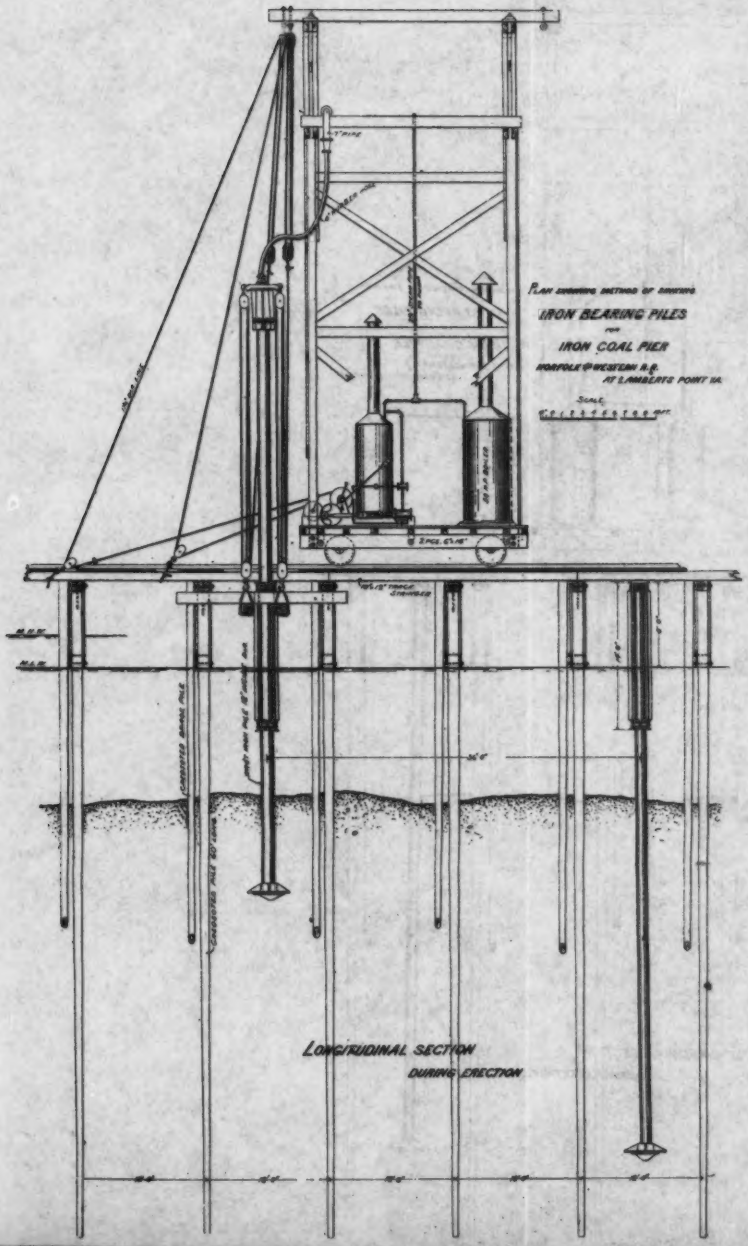
IRON COAL PIER

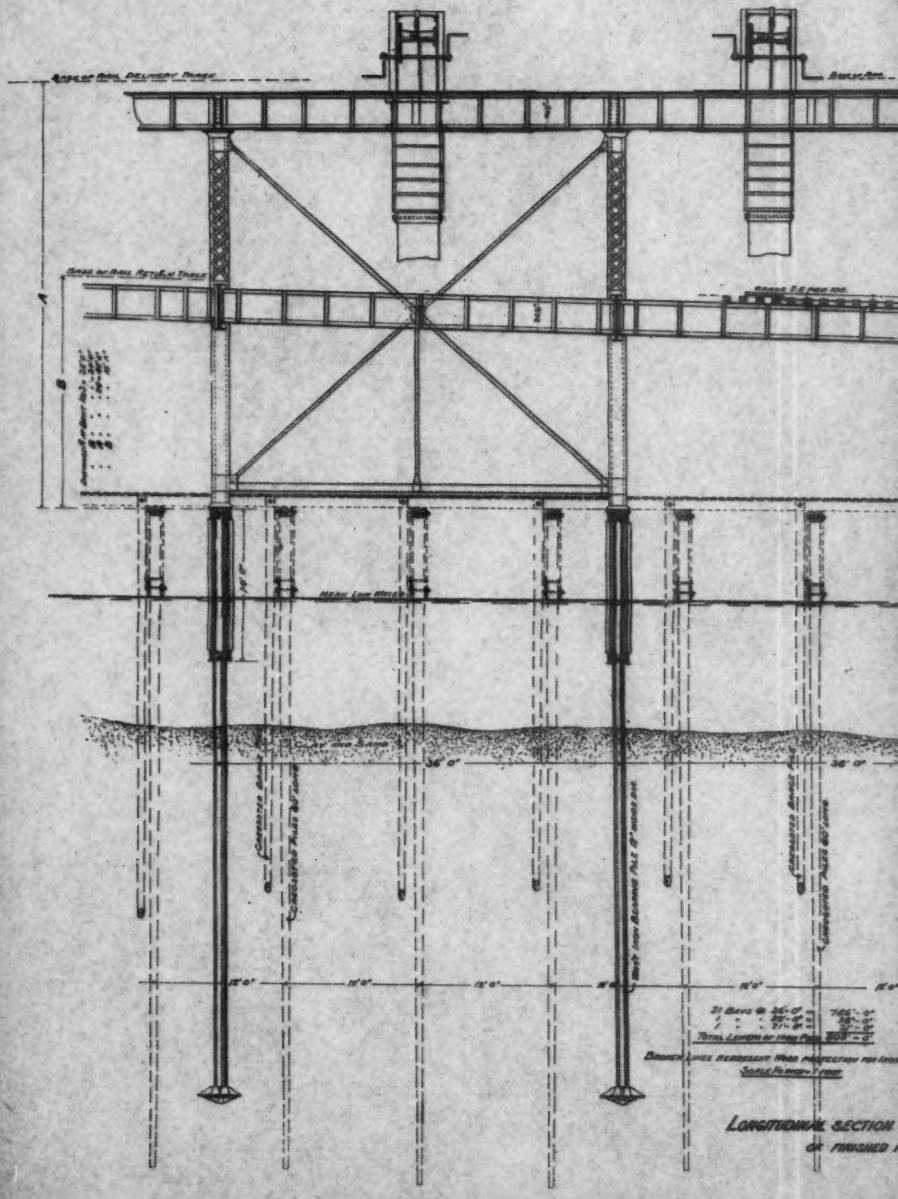
NORFOLK & WESTERN R.R.

AT LAMBERTS POINT VA.

SCALE

PLATE XXIV.
TRANS. AM. SOC. CIV. ENG'RS.
VOL. XXVII. No. 544.
COE ON IRON COAL PIER AT
NORFOLK, VA.





LONGITUDINAL SECTION

PLATE XXV.
TRANS.AM.SOC.CIV.EN
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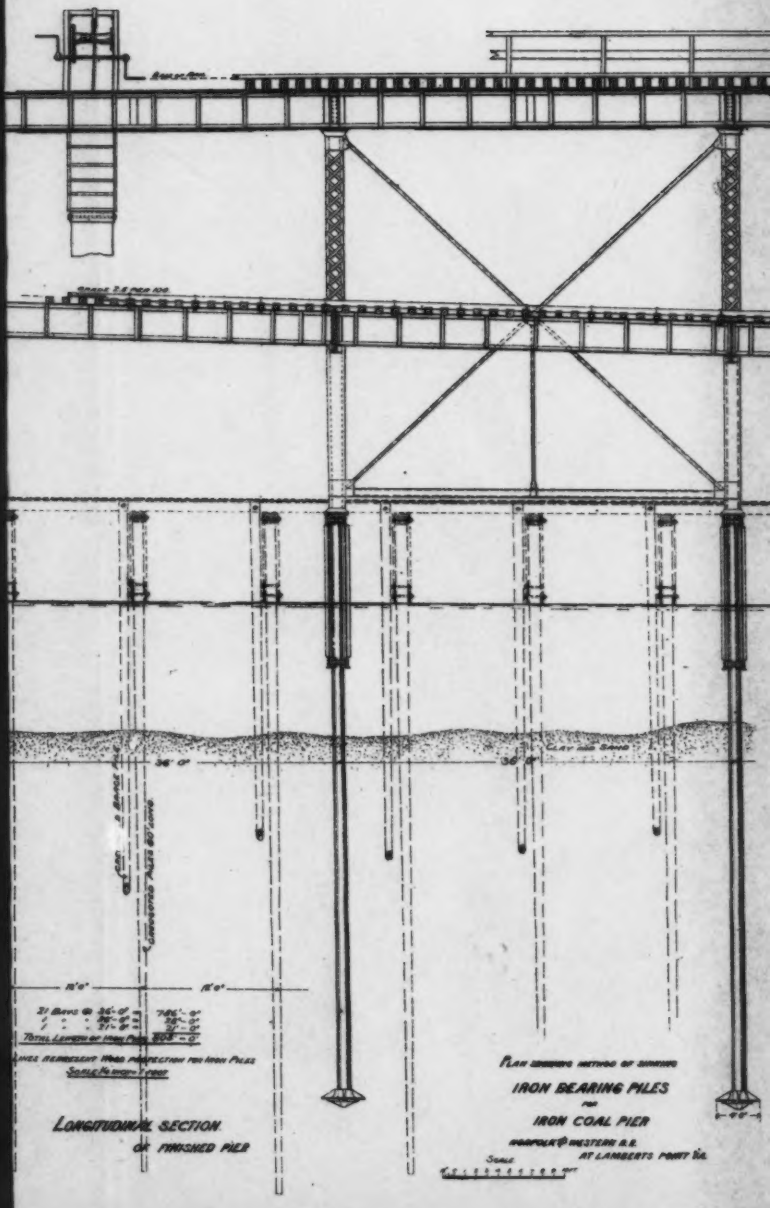
COE ON IRON COAL
NORFOLK.VA.



LONGITUDINAL SECTION
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PLAN SHOWING METHOD OF DRIVING
IRON BEARING PILES
FOR
IRON COAL PIER
NORFOLK & WESTERN R.R.
AT LAMBERTS POINT VA.
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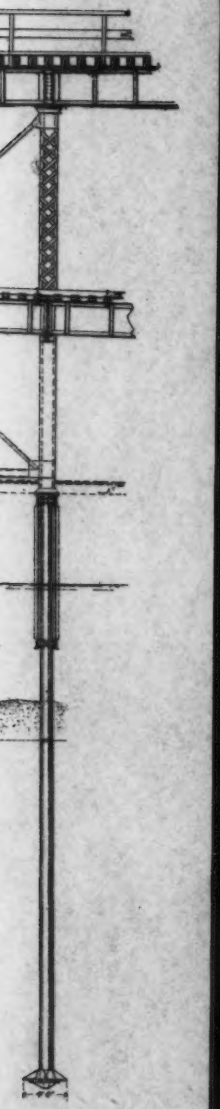

PLATE XXVI.
TRANS. AM. SOC. CIV. ENGS.
VOL. XXVII, No. 544.
COE ON IRON COAL PIER AT NORFOLK, VA.

DISCUSSION.

A. M. WELLINGTON, M. Am. Soc. C. E.—As the author of the formula which Mr. Crowell has been good enough to select as on the whole the most convenient, perhaps I ought to open this discussion; but I must do so more briefly than I should wish, in order to duly cover all points to be considered, as what I have to say would perhaps be more appropriate in the form of a separate paper than in the form of a discussion.

I should premise that in any formula in regard to pile-driving, it must be assumed that the piles are driven under rational conditions, so that they are neither so heavily struck, and so neglected in respect to sawing off broomed heads as to be battered and broomed unreasonably at the time the last few blows are given, nor so lightly struck that the blow does no work. It must always be assumed, too, that the pile has been sinking with reasonable regularity under the last few blows, and that the behavior of the pile when driving, and of the other piles around it, has been such as to give reasonable assurance that the apparent uniformity of set is not deceptive, *i. e.*, that the pile is not really on the point of breaking through a thin crust into a semi-fluid substratum, or anything of that kind. Difficulties of that kind must be guarded against by the judgment and experience of the engineer, or not at all. Formulas cannot attempt to provide for them, but fortunately they are the exception.

The formula to which Mr. Crowell gives the preference was originally published in *Engineering News* of December 29th, 1888, in connection with an extensive discussion of the whole subject, and in comparison with sixteen other formulas. It was there given in the form:

$$L = \frac{2wh}{s + 1},$$

in which $L =$ "the safe working load for piles under all ordinary conditions, to be reduced under exceptional conditions (as notably with irregular penetrations), but never exceeded unless the pile is known to rest on rock and act as a column"; $w =$ the weight of the hammer in pounds; $h =$ the height of fall in feet, and $s =$ penetration under the last blow in inches, "assumed to be sensible, and at an approximately uniform rate."

This formula was put forward as a purely empirical one, and its usefulness established only by comparing its results with known instances of resistance and with the indications of other formulas, but as a matter of fact it was not purely empirical by any means; on the contrary, the general form—

$$L = \frac{fwh}{s + c}$$

was first deduced as the correct form for a theoretically perfect equation of the bearing power of piles, barring some trifling and negligible elements to be noted; and I claim in regard to that general form that it includes in proper relation to each other every constant which ought to enter into such a theoretically perfect practical formula, and that it cannot be modified by making it more complex, as Mr. Crowell proposes, or by making it less complex, without making it less correct and trustworthy as well as less convenient. The numerous formulas by such high authorities as Weisbach, Rankine, Sanders, Nystrom and many others, which contain from four to six other constants or variables, and have much more complex forms, I claim to be not only practically unsuitable and, for the most part, untrustworthy, but theoretically erroneous; not, of course, in their mathematical work, which I should not venture to question, but in their practical premises. This position I shall not now attempt to establish affirmatively, otherwise than by summarizing the process by which my own formula was deduced, which I believe to leave no gaps open for correction by more minute work. An error of judgment which has unnecessarily and greatly complicated many of the formulas is dealing with the energy of the blow in terms of weight multiplied by velocity instead of foot-pounds of work.

In any case of pile-driving by blows, the work done is represented by the product of weight of hammer multiplied by height of fall $= wh$. There is a small loss from air resistance and friction in the guides, but this is infinitesimal with vertical guides, and is universally neglected. Mr. Crowell suggests determining this loss by chronograph observations, but it would be a waste of time. Even with a 30-foot fall the striking velocity is only 30.1 miles per hour; the mean air resistance to the hammer is 2 to 4 pounds, and the loss therefrom merely equivalent to so much off the weight of the hammer, or about 1 part in 1 000. As for friction, that is always a function of pressure, and though the hammer itself is heavy, it can never press with much force against the vertical guides, which it loosely fits, if all is in good order. Even if the guides are not quite vertical, the loss must be small, except in some rare cases when the guides are purposely set much inclined in order to drive inclined piles. Such cases are exceptions not now considered, for which a proper special allowance can be made by the engineer.

All these foot-pounds of energy wh, then, barring this universally and justly neglected fractional loss, are expended somehow during the set s of the pile, so that the mean resistance in pounds to the hammer during that set is expressed by $\frac{wh}{s}$.

If h be in feet, however, and s in inches, this formula takes the form

$$\frac{12wh}{s}.$$

Now, the energy of the blow may be absorbed in any or all of these five ways, which we will severally consider:

1. In brooming and mashing the pile, either (*a*) visibly on the head, or (*b*) invisibly at the foot of the pile, or even in its middle, which latter is far from uncommon.

2. In bouncing, and thus striking two or more light blows instead of one heavy one.

3. In compressing elastically the material of the pile and hammer.

4. In overcoming the inertia of the pile and the static grip of the earth upon it.

5. In causing the pile to penetrate against the surrounding earth's resistance.

1 (*a*). Brooming of the head is an enormous source of loss of work, whenever it occurs, both directly and in cushioning the blow. Some such brooming almost always occurs. The recognized remedy against this, however, and an entirely effective one, is to saw off the broomed part, and give a fresh solid surface for the blow, when the pile is nearly home. Don J. Whittemore, M. Am. Soc. C. E., contributed a paper to this Society some years ago, giving the results of some most valuable observations on this head, which should be studied in this connection.

1 (*b*). Brooming at the foot does not diminish the energy of the blow on the pile, but merely dissipates it unproductively. It can generally, but not always, be detected by the skilled pile-driver by a change in the behavior of the hammer and pile. No formula does or can provide against it. The judgment of the engineer and his assistants must be relied on to detect its probable or actual occurrence, and to act accordingly by stopping the pile-driving after brooming has begun. If all these precautions be taken, brooming will not occur to any injurious extent to any considerable number of piles. Therefore, no formula does or can make any allowance for its effects other than in the usual coefficient of safety.

2. Bouncing invariably means that all the energy of the blow cannot be utilized in work, either because the pile has struck a solid obstacle to further penetration, which is soon detected, or because the hammer is too light or its striking velocity too great, or both, to get the pile in motion before it reacts elastically with more force than the hammer is exerting to push it down. In many cases the remedy indicated is a heavier hammer. In all cases the height of the bounce, and considerably more, represents so much absolutely wasted energy, which goes to increase the brooming but not to increase the penetrating force, because the energy escapes again from the pile before the latter begins to move. The judicious pile-driver, therefore, will and does cut down the fall at once, when he finds that bouncing occurs to any extent, and usually acomplishes far more with the lighter fall by

so doing. A slight bounce at the end of every blow, as we shall see, is an invariable and necessary accompaniment of all good pile-driving.

3. Elastic compression within the limits of elasticity is not a source of loss except to a trifling extent determinable as follows: it is necessarily greatest at the instant after impact. As the pile gets under way, the elastic compression necessary to transmit from the hammer through the pile a pressure sufficient to cause the pile to penetrate, diminishes, but so long as the hammer and the pile together are moving downward, this decrease and resulting elastic expansion all takes effect to cause the foot of the pile to move a little faster than the head and the hammer above it. After the energy of the blow is so spent that the pile can penetrate no further, the remnant of elastic compression in the pile can only expend itself upward, and it then causes a slight bounce of the hammer. Except that the speed of expansion is not quite quick enough it would, we may well believe, cause a considerable bounce, as it does in any case at times.

This last remnant of energy is wasted, but it also is small and negligible, except in enforcing the necessity of a coefficient of safety. None of the formulas which nominally take into account the elasticity of hammer and pile really allow for it in this way. Therefore, except for the proviso hereinbefore considered, all that we need to take into account in seeking to determine some measure of the bearing power of piles, is the work done in actually moving the pile. (Items 4 and 5 above.) This I have dealt with as follows:

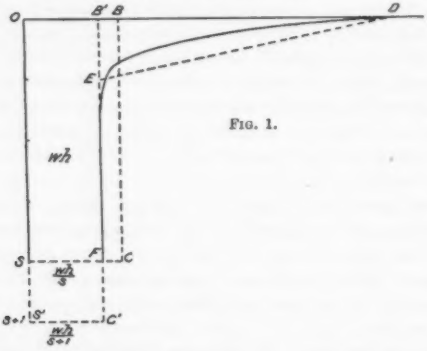

FIG. 1.

Let the ordinate os (Fig. 1) = the penetration of any given pile under a blow, and let the rectangle $osCB = wh$ = the energy in foot-pounds of that blow. Then will the abscissa sC = the mean resistance of the pile during its set. In what manner is that resistance actually distributed, and what part of it bears the nearest relation to the future bearing power of the pile?

So far as the inertia of the pile is concerned, that does not enter into the problem, except as affecting the elastic compression of the pile, which we have already considered. The energy absorbed in acquiring velocity is given out again in surrendering it, and the mean result is the same. What we are after only is the external resistance to the motion of the pile.

In its absolute amount, this is as variable as the earth itself. But reasoning from the general laws of friction and the known nature of earth, these general conditions must obtain: at the instant after impact there must be a very large excess of external resistance, oD (Fig. 1), due to two permanent causes: (1) the excess in the coefficient of static friction or of friction at very low velocities over that at relatively high velocities, and (2) the settling of the earth around and into the irregularities of the pile, in the interval since the last blow. This last, in fact, may be considered as a general phenomenon, which occurs, in some degree, with all surfaces in frictional contact, so that (2) is only a part of (1).

Under the general laws of friction between such substances, as I understand them, this heavy excess of initial friction, oD (Fig. 1), will rapidly, but not instantaneously, decrease to some point E, where it will have a much-reduced value, $s'c'$, which thereafter will remain sensibly constant during the remainder of the set. In this case, we have the irregular figure $oDEFs$, whose area must $= wh$.

The nearest measure of the future bearing power of the pile, obviously, will be the comparatively uniform frictional resistance to penetration, sF (Fig. 1), *after* the initial excessive resistance has been overcome; because, in large part, this excess is due to the suddenness with which the blow acts on the pile, and a gentler unceasing pressure of much less force in pounds may overcome it; besides which, tremors, seepage of water, yielding of surrounding soil, and other like causes may overcome it. The same causes may partially overcome the smaller constant resistance sF, but that possibility we allow for in the coefficient of safety. All that is claimed is, that this force, sF, comes the nearest to a true measure of future bearing power of all possible constants.

To determine sF in terms of non-variables, we must construct a rectangle $os'C'B'$ whose area $= oDEFs = osFB' + DB'E = wh$, which I do as follows:

I assume that the decreasing excess resistance outlined by the line ED, and whose value in foot-pounds is expressed by the irregular area $B'ED$, is confined within the first inch of penetration (*i. e.*, that $B'E = 1$), and that the initial excess $B'D = 3\ oB' = 3\ sF$. The selection of these particular constants instead of others a little higher or lower is pure assumption, in the sense that we can never know experimentally, or at least, do not now know, just how this is. But it is based on exten-

sive observations of the behavior of piles in driving and on many years' experiment and study as to the general laws of friction, as embodied in part in my paper on that subject (*Transactions* Am. Soc. C. E., 1884, Vol. XIII, p. 409), where the diagram of actual observations on page 420 will be found to bear a striking resemblance to Fig. 1 of this discussion. I have no doubt that the above assumptions are essentially accurate and suitable, especially as considerable modifications thereof do not essentially modify results. That the general nature of the distribution of force is correctly outlined in Fig. 1, does not in my judgment admit of doubt.

Under these two assumptions we have geometrically—

$$\begin{aligned}\text{Triangle } B'ED &= \tfrac{1}{2}\, B'D \times B'E \\ &= \tfrac{1}{2}\,(oB' \times 3) \times 1 \\ &= 1\tfrac{1}{2}\, oB'.\end{aligned}$$

But, furthermore, the irregular area $B'ED$ is geometrically equal to about two-thirds of triangle $B'ED$, whence—

$$\text{Irregular area } B'ED = oB' \times 1.$$

Then, in order to determine the value of the abscissa $oB' = sF = s'\,c'$, we must add to the rectangle oF an area $= oB' \times 1$, which we do by making $ss' = 1$ and $s'\,c' = oB'$, whence we have, letting $oB' = R =$ the assumed maximum bearing power of the pile—

$$wh = R \times (s + 1);$$

whence we have—

$$R = \frac{wh}{s+1};$$

or, if h be in feet and s in inches, as usual—

$$R = \frac{12wh}{s+1}.$$

This was and is my formula for the probable ultimate bearing power of piles. Barring the petty exceptions noted above, I believe it to be incapable of further refinement and amplification in any way in order to express more accurately the probable ultimate bearing power; and I believe it to be especially unwarranted and injurious to tamper with the constant 1, as Mr. Crowell proposes, in order to allow for future conditions of the pile in respect to tremors, or in respect to the importance of stability. That constant has no relation to such allowances, any more than the variables w, s or h. The proper way to allow for these future differences of conditions, and the only proper way, is by varying the coefficient of safety.

For myself, I assumed the factor 6 as the coefficient of safety, giving to the formula the form—

$$S = \frac{2wh}{s+1}$$

in which $S =$ the safe load in the pile $= \frac{1}{6}$ the probable ultimate. I assume 6 as a factor of safety, rather than 4 or 5, simply because I

found 6 to give as big loads as are customary, or as are in many cases safe. I do not question that in many cases a factor of 2, 3, 4 or 5, or even 1, might also prove safe. In some cases they have done so, but in other cases they have not; whereas, I have as yet discovered no cases where the factor 6 had proved insufficient, either experimentally or in service. As the factor 6, moreover, will in no case require piles to be driven closer together than is customary and reasonable, I should advise in all cases adhering to it, unless under some very exceptional circumstances, where the engineer may see that he has special cause and justification for taking more chances. He can be reasonably sure that his pile will do its allotted work safely with a factor of 6, but not with a smaller factor—that is all.

Apart altogether from the special reasons so far advanced, it is wrong in principle to tamper with the constant 1 and make it a variable, in order to vary the factor of safety to suit future conditions, as Mr. Crowell proposes. That constant is a part of the denominator of a fraction. For well-understood mathematical reasons, if it is desired to increase the value of a fraction by 10, 20, 50, 100 or any other per cent., which is what Mr. Crowell is aiming at, it is impossible to do so in any simple way by modifying the denominator. It should be done by a factor modifying the fraction as a whole, *i. e.*, multiplying the numerator. When, as in this case, the denominator consists of two terms, any attempt to vary the value of the fraction to suit any desired coefficient of safety, by varying only one of the terms of the denominator, is particularly objectionable.

A further reason why the constant 1 should not be tampered with, is that, as it happens, in the extreme case when $s = 0$, *i. e.*, when the pile has struck solid ground and can penetrate no further, the formulas reduce to the form $R = 12wh$, and $S = 2wh$, which happen to be just about what the pile can safely carry as a column, as ultimate or safe loads, under ordinary and probable values of w and h, whether regarded as a short column with its bearing power limited only by the crushing strength of the wood, or as a long column liable to flexure. Therefore, it becomes unnecessary to consider the special case of the pile acting as a column. The formula is general for all conditions. When the constant 1 is reduced to considerably less than 1, as Mr. Crowell proposes, the apparent safe loads become excessive, and considerably more than the crushing strength of the pile itself. As it is only in these cases that Mr. Crowell's proposed modifications give materially different loads from the original simple form of the formula, it will be seen that the added complexity is not only without advantage, but distinctly injurious.

To prove this fact I have computed the following table, which sufficiently explains itself. The last four columns give the pith of the argument. In these, the last three columns give the ratio of the safe

load indicated by three cases of Mr. Crowell's modified formula in comparison with my original simple form. The table may be divided horizontally into two main parts, that for penetrations of less than 1 inch and of over 1 inch.

Considering the small penetrations first: Mr. Crowell's modifications have substantially no effect on piles for machinery foundations, nor on any piles where the penetration is as much as one-half inch. Less penetrations than that are generally bogus, as practical men know, and indicate that the pile is brooming and mashing rather than penetrating further. For these cases Mr. Crowell's modifications indicate that from one and one-half to three times greater loads are safe, but this is untrue as a fact, as just stated, for the simple reason that such loads amount to one-half to three times the safe strength of the pile as a column.

For penetrations above 1 inch, Mr. Crowell's formula gives almost identical results for "ordinary cases" in a much less simple way; but for railway bridges he says, in effect, that my simpler form gives 2 to 8 per cent. too great loads, and for machinery foundations he says, in effect, that my form gives 10 to 12 per cent. too great loads. These are small differences to dispute over; but I deny the fact that they exist, or that there is any evidence to show that they exist, or tending to show it. Certainly Mr. Crowell advances no evidence to show that they do. I think he will be surprised to find how little effect upon

Last penetration in inches = S.	Factor by which wh is multiplied to give safe load in—				Relative safe load indicated; Engineering News formula taken as 1 000.			
	Original Engineering News formula.	Suggested elaboration by J. F. Crowell.						
		Ordinary cases.	Railway bridges.	Machin'y foundations.	Engineering News.	Ordinary cases.	Railway bridges.	Machin'y foundations.
None ..	1.778	5.480	3.333	2.000	1.00	3.08	1.87	1.13
0¼.....	1.600	3.333	2.500	1.818	1.00	2.08	1.56	1.13
0½.....	1.333	2.100	1.734	1.290	1.00	1.58	1.30	.97
0¾.....	1.143	1.562	1.348	1.064	1.00	1.37	1.18	.93
1.....	1.000	1.250	1.111	.909	1.00	1.25	1.11	.91
2.....	.667	.713	.6515	.587	1.00	1.07	.98	.88
3.....	.500	.504	.480	.438	1.00	1.08	.96	.88
4.....	.400	.392	.377	.351	1.00	.98	.94	.88
5.....	.333	.322	.3115	.293	1.00	.97	.93	.88
6.....	.286	.273	.266	.2525	1.00	.95	.93	.88
7.....	.250	.238	.232	.222	1.00	.95	.93	.89
8.....	.222	.210	.206	.197	1.00	.95	.93	.89
9.....	.200	.189	.185	.1786	1.00	.94	.93	.89
10.....	.182	.171	.168	.163	1.00	.94	.92	.90
11.....	.167	.157	.154	.150	1.00	.94	.92	.90
12.....	.154	.1446	.1425	.1385	1.00	.94	.92	.90

the final result his apparently considerable modifications of the original simple formula really have. Before proposing them, evidence should be advanced to show that the simpler formula is really the less trustworthy of the two in some one case, at least; and, assuming that evidence to have been produced, the proper way to give it effect is to make the coefficient 2 of the numerator a variable, and not the constant 1 of the denominator.

I must not be understood to take the position that the load placed on piles should not be decreased in important structures subject to tremor, or that it may not be increased for carrying unimportant temporary structures. I merely say that any excess or deficit in such cases is guesswork, pure and simple, and therefore does not belong in a formula. The way to make such changes is to space the piles a little nearer together or farther apart than we otherwise would; not to "monkey with" our basic formula (which is already as exact as theory permits us to make it) by arbitrary changes in the general direction of the allowances we wish to make.

JOHN C. TRAUTWINE, Jr., Assoc. Am. Soc. C. E.—"The absence of anything like uniformity in the adaptation of piles to foundations" is indeed lamentable, but I cannot agree with Mr. Crowell that it is surprising. Truly, as he says, "the general principles involved are neither obscure nor unrecognized," but the elements attending the operation of those principles are so uncertain and so numerous, and the experimental data at hand are so uncertain and so few, that the wonder seems to be, not that we are without established and reliable rules, but rather that any one has ventured to formulate any rules at all.

In the case of iron bridges the strains due to the load may be accurately determined; the materials of the structure may be tested by delicate laboratory experiments, not only upon small specimens which reveal more or less clearly the molecular structure of the material, but also upon full-sized and built-up members, repeating almost exactly the conditions of the parts of the actual structure; and the records of such tests, made under private, corporate and governmental auspices, are at hand by thousands. Yet, as Mr. Crowell intimates, candor compels us, even under these circumstances, to call our factor of safety a "factor of ignorance." What term, then, can we find to fittingly express the cimmerian darkness in which we grope when we come to estimate the bearing power of a timber pile driven into the earth by blows of a ram and afterwards subjected to a static or dynamic load?

The weight of ram and the height of fall may indeed be determined with reasonable exactness, and the product of these gives us, no doubt, quite approximately, the energy stored in the ram at the instant when it comes into contact with the head of the pile; but in estimating how much of this energy is utilized in driving the pile, we are obliged to

remember that the mass of the pile has yet to be set in motion; that the direction of the blow can seldom be more than approximately in line with the axis of the pile; that the vibration of the pile under the blow will vary with the length of the pile projecting from the ground; that according to circumstances this vibration may facilitate or impede the driving, and that to an unknown extent; that the pile, even at its best estate, is more or less compressible, and that the head must become more or less crippled by the blows delivered upon it, thus further increasing and rendering still more uncertain the amount of the useless consumption of energy in the delivery of the blow.

But perhaps the most serious difficulty is encountered when we come to deduce the supporting power from the estimated effective energy of the blow, dividing the latter by a quantity containing the penetration or set per blow, as is done in all the formulas embraced in the present discussion. As observed by Mr. Rudolph Hering,* "the only method which can be depended on in calculating the sustaining power of piles held by friction, is the experimental one which introduces the actual distance (s) which a pile sinks under the last blow." And yet this very factor, the absence of which would render these formulas irrational and without which they would fall to the ground, constitutes at the same time, perhaps, their weakest point, especially in formulas which, like Major Sanders', have not a second additive quantity in the divisor. The penetration, as compared with the height of fall of the ram, is in most cases very small. The circumstances of actual practice seldom permit an accurate measurement, even of the *apparent* penetration, which again is always complicated to a considerable and uncertain extent by the compressibility of the pile and of the soil, and by the brooming of the head of the pile. And yet a very small error in the measurement or estimation of the penetration evidently causes a wide divergence in the results given by any formula in which this factor performs a prominent part.

Finally, we are almost entirely without experimental knowledge of the subject. All of the results I have been able to obtain in which the ultimate load is given or from which it may be inferred, are embraced in the handful mentioned below, and some of these (as explained in place) are of an exceedingly doubtful character.

But the uncertainties of the case do not cease with the driving of the pile. Vibrations due to the load itself or to those upon neighboring structures, the lubricating effect of infiltrating streams of water, the reduction of pressure by the dredging away of material from adjoining works; these, and perhaps other causes, may operate to reduce the frictional resistance of the pile.

The author has earned the thanks of the profession by endeavoring to reduce the element of uncertainty attending pile-driving operations.

* "Bearing Piles." Published by *Engineering News*, 1878.

He has done this by selecting that one of the existing formulas ($L = \frac{2wh}{s+1}$) which most commends itself to him by its simplicity and by its conformity to known principles, and has modified it by changing its divisor $(s + 1)$, substituting first the expression $s + 0.3$, and finally the expression $s + 0.1 + n + n'$, in which $n =$ half the square root of the penetrations under a standard blow of 40 000 foot-pounds, and n' serves the purpose of an additional factor of safety and varies from 0.1 to 1 according to the duty to be performed by the structure.

The five formulas discussed by Mr. Crowell may be written thus:*

Sanders:† $L = \frac{1}{8} \cdot \frac{12wh}{s}$‡

Wellington: $L = \frac{1}{6} \cdot \frac{12wh}{s+1}$‡

Crowell (a): $L = \frac{1}{6} \cdot \frac{12wh}{s+0.3}$‡

Crowell (b): $L = \frac{1}{6} \cdot \frac{12wh}{s + 0.1 + \frac{1}{2}\sqrt{s\frac{40\,000}{wh}} + n'}$‡

Trautwine: $L = (\frac{1}{2} \text{ to } \frac{1}{12}) \frac{50w\sqrt[3]{h}}{s+1}$

in which, as in Mr. Crowell's paper,

$L =$ the safe load on one pile;*

$w =$ the weight of the ram;*

$h =$ the height of the fall in feet;‡

$s =$ the penetration per blow under the final blows, in inches;‡

$n' =$ a term varying from 0.1 to 1.

As thus written, the (numerical) fractional coefficient is the factor of safety, and the other factor is the extreme load R on one pile.

It will be noticed that there is a certain similarity of form existing between all of these formulas. All have in the numerator the product of the weight w of the ram by a function of the height h of fall, and in all but ours this function is the fall h itself. They thus agree in deducing, more or less directly, the work done from the energy stored in the ram, without making any definite provision for the inertia of the pile; resembling in this the simplest possible rational formula:

$$R = \frac{12wh}{s},$$

* L and w to be measured in one and the same unit, as both in pounds or both in tons

† Mr. Crowell ascribes this formula to Weisbach, but I do not find it given by that author. It is given by Major John Sanders in the *Journal of the Franklin Institute*, November, 1851, p. 304.

‡ If h and s are taken in one and the same unit, as both in feet or both in inches, the number 12 disappears from the numerator of the first four formulas.

in which R is the resistance (assumed to be constant) of the earth to the penetration of the pile; in other words, the extreme load of the pile.

Like this very simple formula, also, all of those discussed contain in their denominators the penetration, s, of the pile under the last blow, or, as it is often put, the penetration per blow under the last few blows; but, in all except that of Major Sanders, the denominator contains a second term to be added to the penetration, and in Mr. Wellington's formula and our own this added term is simply 1 inch.

Mr. Crowell refers to the use, in our formula, of the cube root of the fall as being theoretically illogical and practically inconvenient. Inconvenient it certainly is, as compared with the use of the fall itself, but I do not see that it is "theoretically illogical." On the contrary, so long as increase of height of fall beyond moderate limits is found to be attended by increased losses of energy, it must be theoretically logical to make allowance for them, and the simplest if not the most rational way of doing this would seem to be, to represent the energy of the blow as varying with some power of the fall lower than the first.

Other things being equal, a high fall seems to cause a greater waste of energy in damaging the pile, and, as Mr. Wellington has pointed out, in rebounding of the ram, besides allowing a longer time for the material around the pile to re-compact itself.

McAlpine's experience at Brooklyn led him to believe that the supporting power of a pile varies as the *square* root of the fall.

It can readily be shown that the large coefficient (50) necessitated by our use of the cube root gives excessive loads in cases of very low falls, such as may perhaps occur at times with the Nasmyth steam-hammer pile-driver. Thus, for a fall of only 1 foot, our $50\sqrt[3]{h}$ becomes $50h$, and may thus give a load even much greater than $\frac{12wh}{s}$ (see Fig 2). For a 2 000-pound hammer falling 1 foot and producing 1.5 inches penetration, our rule gives 40 000 pounds as the extreme load, whereas the greatest theoretical extreme load $\frac{12wh}{s}$ is only 16 000 pounds. It may be proper to consider, however, that such cases hardly come within the actual probabilities of pile-driving, and these alone our formula was intended to cover. Indeed, I have no doubt it was constructed by its author as a rule of thumb by making it fit such experimental cases as came within his observation, and it is safe to assume that none like the one supposed were encountered by him.

Whatever may be thought of Mr. Wellington's demonstration of the correctness of the additive term 1 in the divisor of his formula and in that of ours, the omission in Sanders' formula of any such term must, I think, be regarded as a fatal defect, for it makes that formula give excessive loads for very small penetrations, and infinite loads in cases where no penetration can be detected, and this no matter

how light the ram or how low the fall. This, of course, is theoretically correct. A pile which undergoes absolutely no set under a blow from a falling feather will theoretically support the universe, but a formula which would make it do so in practice is hardly

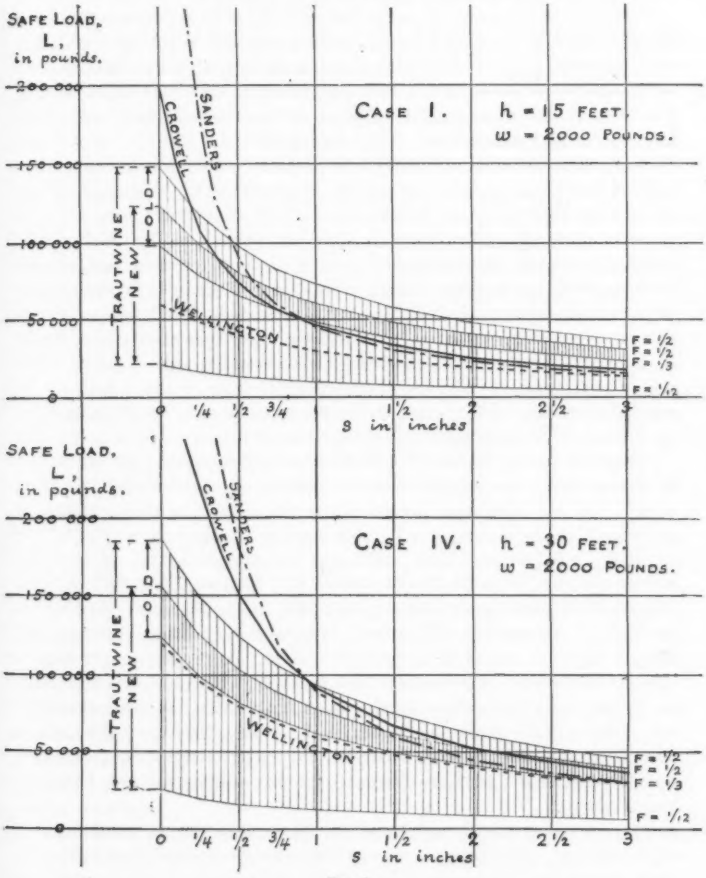

FIG. 1.

applicable to cases of very small penetration. It is but just to Major Sanders to note that in proposing this formula he limits its application to cases where the pile "meets such an uniform resistance as is indicated by slight and nearly equal penetrations for several successive

blows of the ram." This rules out cases where the observed penetration is 0, but still admits those where the penetration is very small and where the safe load by the formula is therefore excessively great, as indicated in Fig. 1 herewith.

It has always been my impression that the "1" in the divisor of our formula was simply an empirical term used by the author to avoid the difficulty just referred to, and that he arrived at its value by trial with recorded cases.* Mr. Wellington's claim that it can be shown to be logically correct and a necessary factor in any rational formula, was, therefore, an agreeable surprise to me, and I have read with much interest his argument in the premises.

But, although I am naturally willing to be persuaded of the soundness of this position, I am obliged to confess that the perusal has not resulted in that firmness of conviction which could have been desired. It seems perfectly reasonable to assume that there is, as Mr. Wellington claims, an excess of resistance at the beginning of penetration; but we are not shown how the values of $B'D$ and $B'E$ in Mr. Wellington's Fig. 1 ($B'E = 1$ inch, and $B'D = 3.OB'$) are deduced from his observation of the behavior of piles, and from his experiments and study as to the laws of friction, and in the absence of this I am inclined rather to endorse his admission that the adoption of these values "is pure assumption, in the sense that we can never know experimentally, or at least do not now know, just how this is."

Hence, I cannot join in Mr. Wellington's objection to Mr. Crowell's innovation in "tampering with the constant 1, and making it a variable." On the contrary, it seems to me altogether probable that this term may, in fact, vary greatly (perhaps as much as from 0 to 2, 3 or 4 inches, or even more) with differences in the character of the soil and of the pile, or in the weight of ram and height of fall, and should, therefore, be made to appear as a variable in the formula. Indeed, if we had at command a sufficient assortment of reliable experimental data, I think we might be able, by modifying this term, to produce a more nearly serviceable formula than any yet presented. Mr. Crowell's proposed innovation, therefore, so far from being an unpardonable sin, seems to me to be an effort in the right direction, although a comparison of his results with the few experimental data at hand seems to indicate that he has not yet hit upon just the happiest combination.

It appears to me also, in view of "the excess in the coefficient of static friction, or of friction at very low velocities over that at relatively high velocities," that Mr. Wellington's diagram should show an

* In presenting his formula, he remarked: "Although this is all wretchedly empirical, it certainly appears to accord moderately well with such facts as we have been able to obtain. Like other rules of this kind, however, it should be used with caution; and with a wide margin for safety in important cases."

increase of resistance toward the *end* of the penetration, where the pile is being brought to rest, and where, consequently, the velocity is rapidly decreasing. This final excess of resistance would, no doubt, be less than the initial one ($B'ED$); but, such as it may be, it would seem sufficient to render us cautious in accepting implicitly an argument based upon the diagram without it.

Mr. Crowell, in his first suggested modification of Mr. Wellington's formula, proposes substituting 0.3 in place of 1.0 in the divisor. That this is an unfortunate deviation in the direction of Major Sanders' omission of any additive term is indicated by my Fig. 1, where the curve representing this form of Mr. Crowell's formula is seen to mount very rapidly as the penetration becomes less than, say, 1 inch. Mr. Crowell's final formula, in which this term becomes $0.1 + \frac{1}{2}\sqrt{s\frac{40\,000}{wh}}$,* suffers even more severely from this defect, for the additive term here diminishes with the penetration. While we may not unquestioningly accept Mr. Wellington's argument in behalf of the constant value "1" for this term, these considerations certainly indicate that it is, at least, safer than Mr. Crowell's proposed modifications.

I am inclined to question the propriety of allowing the factor n to be determined by means of a standard blow of 40 000 foot-pounds, as in Mr. Crowell's formula, without restriction as to weight of ram or height of fall. It sometimes happens that a pile refuses under a light ram with high fall, but moves satisfactorily with a heavy ram and low fall, and sometimes the reverse of this obtains, perhaps, chiefly in cases where a hard and shallow stratum is encountered.

A little consideration will show that the increase in complexity due to Mr. Crowell's modification (*b*) of Mr. Wellington's formula is less than might be supposed at first glance. Both n and n' are obtained once for all at the beginning of any given set of operations—the former by ascertaining the average penetration under the standard blow, and the latter by consulting the table furnished by the author. In the absence of experimental confirmation, however, we can but accept the author's values of n and of n' as his own guesses, and be thankful that he has done the guessing for us instead of leaving us with a fixed factor of safety, as do Major Sanders' and Mr. Wellington's formulas, or with a factor varying between one-half and one-twelfth, as in late editions of Trautwine.

It will be noticed that in all the formulas, except Mr. Crowell's (*b*) and our own, the value of the factor of safety is fixed. In ours, "not more than one-half the extreme load" is to be taken "for piles thoroughly driven in firm soils, nor more than one-sixth when in river mud

* Inasmuch as the term n' of Mr. Crowell's final formula serves the purpose of a factor of safety, and varies according to an arbitrary scale, with the use to which the superstructure is to be put, it should not, I think, be included here.

or marsh"; while, if there is liability to tremors, only one-fourth and one-twelfth of the extreme load are to be taken respectively. Mr. Crowell has undertaken to provide, in addition to his fixed factor of safety of one-sixth, a sliding scale for the term n' in the divisor of his formula, whereby that term is given a series of fourteen different values, depending upon the service for which the superstructure is to be used and upon the exposure to which it is to be subjected.

In view of the great diversity of the conditions under which piles are driven and of those under which they are subsequently employed, I cannot but think it better to have an elastic factor of safety rather than one which remains the same whether the superstructure is to be on the one hand a lumber-yard or a mausoleum, or, on the other hand, a trestle or a draw-span to be used by trains of all descriptions.

It would appear, therefore, most desirable to provide a series of factors for safety, arranged, like Mr. Crowell's, to suit the various conditions of service; but, until the experimental data at our command become vastly more numerous and more reliable than they now are, any such table must, as already intimated, be taken, like the "1" in the divisors of Mr. Wellington's formula and of our own, as a "pure assumption." In the present state of our ignorance, much must be left to the judgment of the engineer, and he may well be thankful if we succeed in concocting a rule of thumb which gives results bearing some sort of proportion to the probable extreme load, leaving it to him to decide what portion of this load he will take his chances with.

Mr. Crowell has kindly corrected a misapprehension into which he (in the advance copy of his paper) had fallen respecting the Trautwine formula, and in which Mr. Hering (in the pamphlet already quoted) had preceded him. Both gentlemen refer to an earlier and a later form of our formula, and in this they have, in a measure, followed the example of its author, who, in the first and second editions of the "Pocket Book," said:

"When the pile has ceased to sink appreciably under a given fall and weight of hammer. Extreme load in tons $= \sqrt[3]{\text{fall in feet}} \times$ weight of hammer in tons $\times$ 60"; and added: "But if the pile sinks appreciably under each blow, but not exceeding a few inches, then divide the extreme load as just found by the sinking in inches, added to the constant 1."

In later editions the formula is made general, being, in effect:

$$\text{Extreme load} = \frac{50 \times \text{weight of hammer} \times \sqrt[3]{\text{fall in feet}}}{\text{penetration in inches} + 1.}$$

It is evident that except for the change in the coefficient from 60 to to 50 (the only change that has been made in the formula itself) this general form is equivalent to that first given, with its provision for cases with appreciable penetration.

The error I have referred to, is that both gentlemen have taken the formula given by the author in the first edition, for cases where there is no penetration, and have applied it to a supposed case with a penetration of 1.2 inches. Thus misapplied, the formula is made to give two results, differing by more than 100 per cent. in the extreme load for a given case, as in Mr. Crowell's original Diagram 1, which was sent out with the advance sheets of his paper, whereas the reduction of the coefficient from 60 to 50* in the third edition makes, of course, a difference of only one-sixth, as shown in Mr. Crowell's corrected diagram exhibited at the Convention.

But, while the form of our equation for extreme load has undergone no change, and while the coefficient C has been reduced only from 60 to 50, our factor of safety has undergone a quite interesting process of development, indicating a growing conservatism on the part of the author of the formula. In the first edition, 1872, in which the coefficient C was 60, the factor of safety was made "about one-third to one-half." In the third edition, 1874, in which the coefficient C was reduced from 60 to 50, it was made "not more than one-half for piles thoroughly driven in firm soils, nor more than one-fourth when in river mud or marsh." In the fifth edition, 1881, the latter figure was made one-sixth, and the remark added: "If liable to tremors, the loads should not exceed one-half of this." And in the seventh edition, 1882, this latter caution is put more positively, thus: "If liable to tremors, take only half these loads."

Comparing, then, our formula as it appeared in the first and in the later editions, we find that in the former we have

$$\text{Extreme load in pounds} = \frac{60 \times \text{weight of ram in pounds} \times \sqrt[3]{\text{fall in feet}}}{\text{penetration in inches} + 1}$$

with factors of safety from one-half to one-third, and that in the later editions the coefficient has become 50 and the factor of safety from one-half to one-twelfth.

Hence, Mr. Crowell's representation of our formulas in his Diagram 2, while correct as far as it goes, is not sufficiently comprehensive to represent our formula properly, for his upper curve represents it with a coefficient of 60, and a factor of one-half, while the lower one represents it with a coefficient of 50 and a factor of one-third. A satisfactory showing would require, in each of his four diagrams, at least four curves for our formula, viz., two with a coefficient of 60, one of them with a factor of one-half, the other with a factor of one-third; and two with a coefficient of 50, one of them with a factor of one-half, and the other with a factor of one-twelfth, and I therefore submit herewith (Fig. 1) diagrams for his two extreme cases, I and IV ($h = 15$ feet, and $h = 30$ feet respectively), drawn in accordance with this suggestion.

* When the load is taken in tons, the weight of ram remaining in pounds, these coefficients become respectively 0.0268 and 0.023.

In these diagrams I have used lines of the same characters as those used by Mr. Crowell to designate the Sanders, the Wellington and the Crowell (*a*) formulas respectively, each of which, allowing no latitude as to the coefficient of safety, is properly represented by a single curved *line*, while our formula, in its two forms, each with a variable factor of safety, is properly represented by shaded *areas*.

These diagrams show clearly the defect of the Sanders formula already referred to, the allowed load rapidly approaching infinity as the observed penetration falls below three-fourths and one-half inch, and they also indicate, I think, that Mr. Crowell's proposed substitution (*a*) of 0.3 for 1.0 in Mr. Wellington's formula, errs in the same direction.

The resemblance between Mr. Wellington's formula and our own is also shown by these diagrams, each of which represents a constant height h of fall, Mr. Wellington's formula falling well within the very wide area embraced between the limits of our later form, and necessarily partaking of its character. But lest it might be hastily assumed, in view of this, that Mr. Wellington's formula was but an imitation of ours, I submit also a second diagram, Fig. 2, in which the penetration s (assumed at 1 inch) is supposed to be constant, while the fall h is made to vary.

Plotted in this way, the formulas of Sanders, Wellington and Crowell (*a*) are necessarily straight-line formulas, as is also the theoretical formula.

$$\text{Extreme load} = \frac{12\,wh}{s}$$

which I have added here for comparison; while our formula, of course, appears as a curve determined by the presence of the cube root of h in its divisor. It will be seen that with a penetration of 1 inch Mr. Crowell's formula (*a*) gives results almost identical with those of Major Sanders' formula. This diagram exhibits also the fact, already noticed, that our formula, owing to its employing the cube root of the fall, gives excessive loads for very low falls.

It is an ominous fact that neither Mr. Hering, in his pamphlet already quoted, nor the author, in the present paper, brings forward any results of experiments in support or in refutation of any of the formulas; ominous because it indicates the dearth of such results. For years I have been on the lookout for data of this kind; but the result, as embodied in the following table, is discouraging in the extreme. I cordially second the hope of the author "that in the discussion which this paper may call out, practical examples will be given, taken from the actual behavior of piles in service, under loads of various character."

When only the actual *safe* load is given, we are left in doubt as to how much more the pile would safely bear. Hence I have restricted

my list of results chiefly to those which give, or from which we may infer, the ultimate or extreme load which causes settlement of the pile and which our author rather unfortunately calls "the sustaining

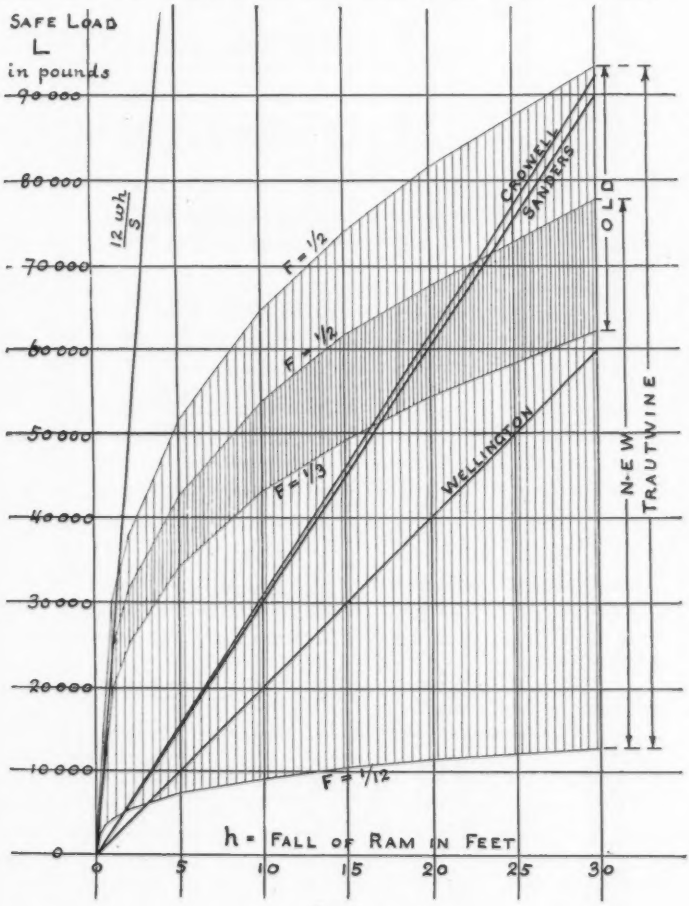

FIG. 2.

power" of the pile. I have omitted cases where the piles are known to have failed by being pushed over laterally by the pressure of adjoining arches, or through similar causes other than the vertical pressure of the loads upon their heads.

TABULAR comparison of experimental results in pile-driving, with the results obtained by Wellington's, Crowell's (*b*) and Trautwine's formulas. See remarks on the several cases, following the table.

A.—PILES TESTED BY LOADING.

No.	Designation.	w. Weight of Ram. Pounds.	h. Height of Fall. Feet.	s. Penetration. Inches.	Extreme Resistance of One Pile, in Pounds. R. Observed or Estimated.	R_w Wellington. $\frac{12\,wh}{s+1}$	R_c Crowell. $\frac{12\,wh}{s+0.1+n}$	R Trautwine. $\frac{50\,w\sqrt{h}}{s+1}$	$\frac{R}{R_w}$	$\frac{R}{R_c}$	$\frac{R}{R_t}$
1	Aquia Creek (*a*)	2 000	4.0	8.5	13 333	10 104	8 094	16 710	1.320	1.647	0 798
		2 000	9.67	22.0	13 333	10 092	9 113	9 261	1.321	1.463	1.439
2	Philadelphia	1 600	36.0	18.0	14 560	36 400	35 000	13 903	0.400	0.416	1.047
		1 600	36.0	18 0	20 160	36 400	35 000	13 903	0.554	0.576	1.450
		1 600	36.0	18.0	33 600	36 400	35 000	13 903	0.924	0.960	2.415
3	Mississippi	1 600	25.0	3.0	22 400	120 000	120 900	58 500	0.187	0.185	0.383
		1 600	30.0	2.5	22 400	164 600	173 500	71 022	0.136	0.129	0.315
4	Perth Amboy	1 700	25.0	2.0	44 800	170 000	183 100	82 ›46	0.263	0.246	0.541
5	Fort Delaware	1 900	6.0	1.0	> 47 375	68 400	67 190	86 300	0.693	0.706	0.549
6	Proctorsville	910	5.0	0.35	62 500	40 446	41 150	57 633	1.545	1.519	1.084
7	Buffalo	1 900	29.0	1.5	75 000	264 480	311 600	116 747	0.284	0.241	0.643
		1 900	29.0	1.5	100 000	264 480	311 600	116 747	0.378	0.321	0.857
		1 900	29.0	1.5	150 000	264 480	311 600	116 747	0.567	0.481	1.285
8	Brooklyn	2 240	30.0	0.	224 000	806 400	8 064 000	348 000	0.278	0.028	0.644
		2 240	30.0	0.2	224 000	672 000	1 705 000	290 000	0.333	0.131	0.772
		6 720	3.0	0.	224 000	241 920	2 419 200	484 579	0.926	0 093	0.462
		6 720	3.0	0.2	224 000	201 600	393 300	403 816	1.111	0.570	0.555
							Mean		0.660	0.571	0.896
							Maximum		1.545	1.647	2.415
							Minimum		0.136	0.028	0.315

9	Dordrecht	2 205	2.5	0.375	13 440	48 120	50 880	108 800	0.279	0.264	0.123
		992	6.6	0.375	13 440	57 120	63 870	67 673	0.235	0.210	0.199
						Mean			0.257	0.237	0.161
10	Aquia Creek (*b*)	2 000	5.0	0.0	2 000	120 000	1 200 000	171 000	0.017	0.0017	0.012
	See note.	2 000	14.0	4.5	2 000	61 100	57 340	43 820	0.033	0.035	0.046
11	Aquia Creek (*c*)	2 000	8.0	1.25	2 000	85 332	86 100	88 889	0.023	0.023	0.022
	See note.	2 000	10.0	2.75	2 000	64 002	59 700	57 451	0.031	0.034	0.035
		2 000	28.0	8.5	2 000	70 740	68 360	31 964	0.028	0.029	0.063
						Mean			0.026	0.025	0.036
						Maximum			0.033	0.035	0.063
						Minimum			0.017	0.002	0.012
12	Lake Pontchartrain	2 500	30.0	3.0	> 22 400	225 000	241 100	97 100	0.100	0.093	0.231
		2 500	30.0	6.0	> 22 400	128 580	128 700	55 500	0.174	0.174	0.404
		2 500	30.0	9.0	> 22 400	90 000	88 240	38 840	0.249	0.254	0.577
		2 500	30.0	12.0	> 22 400	69 228	67 360	29 880	0.324	0.332	0.750
						Mean			0.212	0.213	0.490
						Maximum			0.324	0.332	0.750
						Minimum			0.100	0.093	0.231

B.—Piles Tested by Pulling.

		w. Weight of ram. Pounds.	h. Height of fall. Feet.	s. Penetration. Inches.	Extreme Resistance of One Pile, in Pounds. R. Observed or Estimated.	R_w Wellington. $\frac{12\,wh}{s+1}$	R_c Crowell. $\frac{12\,wh}{s+0.1+n}$	R_t Trautwine $\frac{50\,w\sqrt[4]{h}}{s+1}$	$\frac{R}{R_w}$	$\frac{R}{R_c}$	$\frac{R}{R_t}$
13	Small Piles	22.8	4.6	0.94	490	648	Formula not applicable.	977	0.758	Formula not applicable.	0.503
		24.9	7.6	0.89	572	1 200		1 292	0.477		0.443
		23.8	7.2	0.36	1 288	1 506		1 683	0.856		0.766
		23.2	7.2	0.45	822	1 386		1 550	0.594		0.531
		23.1	7.3	0.67	742	1 218		1 344	0.610		0.553
		39.0	6.8	0.76	959	1 806		2 105	0.532		0 456
						Mean			0.638	.....	0.542
						Maximum			0.856	.....	0.766
						Minimum			0.477	.....	0.443
14	Ice-drawn pile	940	20	0.5	15 850	150 402	202 000	85 051	0.105	0.078	0.186
15	Cambria	350	8	0.125	25 000	29 868	37 580	31 100	0.837	0.665	0.804
		350	8	0.125	37 000	29 868	37 580	31 100	1.239	0.985	1.190
		350	8	0.125	50 000	29 868	37 580	31 100	1.674	1.331	1.608
						Mean			1.250	0.994	1.201

16	Pensacola	2 200	30	0.5	64 000	528 000	905 100	227 867	0.121	0.071	0.281
		2 200	30	0.5	83 000	528 000	905 100	227 867	0.157	0.092	0.364
		4 100	33	0.	64 000	1 623 600	16 236 000	657 537	0.039	0.004	0.097
		4 100	33	0.	83 000	1 623 600	16 236 000	657 537	0.051	0.005	0.126
		4 100	10	0.6	64 000	307 500	454 700	276 032	0.208	0.141	0.232
		4 100	10	0.6	83 000	307 500	454 700	276 032	0.270	0.183	0.301
								Mean	0.141	0.083	0.233
								Maximum	0.270	0.183	0.364
								Minimum	0.039	0.004	0.097
17	Hull	2 240	5	0.75	71 300	76 800	80 480	109 440	0.928	0.886	0.652
		2 240	6	0.60	71 300	107 520	133 300	135 670	0.663	0.535	0.525
								Mean	0.796	0.711	0.588

In a number of the cases recorded, the resistance of the pile was ascertained by *up-lifting* it instead of by loading it. I venture the opinion that, other things being equal, the force required to start a pile upward will usually be less than that required to start it downward, and this notwithstanding that the weight of the pile favors the latter process; for a further driving necessitates a further compacting of the already compacted soil, besides encountering the resistance at the foot of the pile, whereas the up-lifting has merely to break up the cohesion (generally slight) of the upper portions of the material into which the pile is driven.

It must be observed that even the best of these results are to be accepted with more or less reserve; and that their confirmation or condemnation of any particular formula or of all the formulas, is to be taken as an indication merely. The accounts of some of the cases are quite explicit, but in others the attempt to get at the results was handicapped by most aggravating ambiguity in the language used in the descriptions. Where uncertainty exists, it has been indicated in the remarks describing the cases. Some of the accounts state that the heads of the piles were protected by rings, but in others even this information is not vouchsafed, and none of the accounts state distinctly the conditions of the heads, a feature which, as pointed out by Mr. Wellington, is of the first importance in a comparison of the kind here attempted.

In most of the cases we are left uninformed as to the length of time which elapsed between the driving and the testing of the pile, a factor which, as many records show, has often a very marked effect upon the bearing power of the pile, generally increasing it and often to a surprising extent, especially in soft mud (see Cases 10 and 11 and the notes referring to them).

For the purpose of placing the three formulas upon the same footing with the experimental results, I have given for each the extreme load obtained by means of it. This extreme load, in Mr. Wellington's formula, is simply six times the safe load. In Mr. Crowell's it is six times the value obtained from the formula by omitting the term n' from the divisor. Our formula, as it stands, calls for the extreme load.

I have given also the quotients obtained by dividing the actual extreme load R by the extreme load as given by each of the formulas. These quotients, therefore, are the coefficients by which the results of the formulas must be multiplied in order to obtain the actual results. The degree of uniformity of these coefficients, and their approximation to unity, may be taken as an indication of the relative utility of the formulas, always bearing in mind the exceptional character and the extreme unreliability of some of the experiments themselves.

It will be seen that this comparison goes far to strengthen the position of those who claim that formulas are of but little, if any, use in arriving at the supporting power of a pile.

In Cases 1 to 12 the piles were tested by loading; in Cases 13 to 17 by up-lifting. The first eight may be considered good cases; the conditions of driving and loading correspond with what may be called ordinary practice, and in most of these cases are stated with reasonable clearness. In Case 9 the conditions were not unusual, but the data used are open to grave doubt, owing to ambiguity in the printed record.

In Cases 10 and 11 the piles sank under their own weight and that of the hammer, but the sinking took place when the piles were first set up, while the blows recorded were not delivered until four weeks later. In Case 12 the load (very roughly estimated) has been carried for years without settlement. The extreme load is not given.

CASE 1.—Pile trestle at Aquia Creek, Va., 1871 (*a*).* A portion of these results are given in abstract in *Railroad Gazette*, August 12th, 1887, p. 521. Material.—Creek bottom, and almost fluid mud over 80 feet deep. Tide water 6 feet deep. Trestle bents of six piles each, 12½ feet between centers and about 15 feet high. Piles 15 to 18 inches diameter at butt, 50 to 56 feet long, cut off just above low water-mark.

CASE 2.—Philadelphia, 1873. "Trautwine's Civil Engineers' Pocket-book," p. 643. Material, soft river mud. Trial pile loaded with 14 560 pounds five hours after driving, and sank but a very small fraction of an inch. Under 20 160 pounds it sank three-quarters of an inch; under 33 600 pounds, 5 feet.

CASE 3.—Mississippi River at East St. Louis, 1868–69. Letter from Mr. G. Bouscaren, June 25th, 1892. (This case was described by Mr. Bouscaren during the discussion at the Convention.) Material, a soft muddy bottom, with 5 or 6 feet of water. Piles in temporary railroad trestle of three-pile bents 15 feet from center to center. Piles driven about 20 feet. Penetration *s* 2½ to 3 inches. The piles settled badly in a very short time under locomotives weighing not over 30 tons, so that the load on a pile could hardly have exceeded 22 400 pounds. All three formulas give excessive loads in this case.

CASE 4.—Perth Amboy, 1873. Letter from Mr. John F. Ward, July 20th, 1892. Material, "pretty fair mud," 30 feet deep. Four piles, 12, 14, 15 and 18 inches diameter at top, 6 to 8 inches at foot, were driven in a square to depths of from 33 to 35 feet. Distance apart not given. A platform was built upon the heads of the piles and loaded with 80 tons (179 200 pounds) = say, 44 800 pounds per pile. After a few days the load was removed. The 18-inch pile had not moved; the 12-inch pile had settled 3 inches, and the 14 and 15-inch piles had settled to a less extent.

* Letter from Major E. T. D. Meyers, June, 1892.

This case emphasizes the uncertainty arising from the presence of the penetration in the formulas. It is hardly supposable that the same ram, with the same height of fall, should have given precisely the same penetration for piles of such different dimensions and (as the test showed) of different resisting powers, yet the record gives a uniform penetration of 2 inches for all, the available means for measurement evidently failed to detect any difference.

CASE 5.—Fort Delaware, 1850. Memoir of the Life and Services of Captain and Brevet-Major John Sanders, by James St. C. Morton, 1st Lieutenant, Corps of Engineers, Pittsburgh, 1861. The account is highly unintelligible. Material, an alluvial deposit. The piles were of Chesapeake yellow pine weighing from 32 to 59 pounds per cubic foot. They measured 12 inches square, and were about 30 feet long, with points 5 feet long. Four trial piles were driven about 24 feet (distance apart not stated) and capped with a platform upon which the loads were placed. The weight w of the ram is variously given at from 1 883 to 2 000 pounds. It appears to have been about 1 900 pounds.

The loads and settlements were as follows:

LOADS.				SETTLEMENTS.	
On the 4 piles.	On each pile.	Time, mos.	Total time, mos.	Distance, in $\frac{1}{32}$ ins.	Total dis. in $\frac{1}{32}$ ins.
60 700	15 175	6	6	6	6
75 700	18 925	....	....	0	6
94 000	23 500	1	7	2	8
94 000	23 500	3	10	0	8
107 500	26 875	1	11	1	9
121 800	30 450	8	19	7	16
121 800	30 450	33	52	0	16
134 000	33 500				
147 000	36 750	7	59	0	16
160 000	40 000				
174 000	43 500				
189 500	47 375	17	76	10	26
189 500	47 375	48	124	6	32

The total settlement here was so small and so gradual that even the greatest of these loads can hardly be called "extreme"; but inasmuch as this minute detail gives some idea as to *how* safe the piles are under their load, the case is entered in the table, with this reservation, for purposes of comparison.

CASE 6.—Proctorsville, La. From "Van Nostrand's Magazine," July, 1882. Material, mud, sand and clay. Wet. Trial pile (driven alone) said to have been 30 feet long, yet it is said to have sunk 5 feet 4 inches by its own weight, and to have been driven 29 feet 6 inches deeper, making 34 feet 10 inches driven length. Cross-section, $12\frac{1}{4}$ x 12 inches at top, $11\frac{1}{2}$ x 11 inches sharpened to 4 inches square at

foot. Weight, 1 611 pounds. Head capped. Pile bore 59 618 pounds without settlement, but settled slowly under 62 500 pounds. Fall, during last ten blows, regulated to 5 feet exactly. Penetration, last ten blows, ranged from one-fourth to one-half inch; mean 0.35 inch; last blow, three-eighths inch.

CASE 7.—Buffalo. Letter to the writer from Mr. W. A. Haven. Material, wet sand and gravel. Piles driven in nests of from nine to thirteen piles. Test pile of beech, 20 feet long after being driven and cut off. Driven length, 20 feet; 3 feet in stiff clay; cross-section, 15 inches diameter at top. A load of 50 000 pounds remained on the pile for twenty-seven hours, but produced no appreciable effect. The load was increased 20 000 pounds at a time, and left for twenty-four hours after each increase. A gradual settlement aggregating five-eighths inch took place under 75 000 pounds, and the pile then came to rest. The total settlement increased to $1\frac{1}{2}$ inches under 100 000 pounds, and to $3\frac{1}{16}$ inches under 150 000 pounds. During the experiments the ground was kept in a tremor by the action of three pile-drivers at work on the foundations. Subsequent use shows that 74 000 pounds is a safe load.

CASE 8.—Brooklyn, N. Y., 1847–48. Dry Dock at Navy Yard. William J. McAlpine in Procs. Inst. C. E., Vol. XXVII, Session 1867–68, and in *Journal of the Franklin Institute*, 3d series, Vol. LV, Nos. 2 and 3, February and March, 1868. Also, the "Naval Dry Docks of the United States," by Charles B. Stuart, New York, 1852. Material, wet, loamy, micaceous, quartz sand, becoming quicksand wherever it was much trodden.

"The main piles were mainly round spruce spars, very straight, from 25 to 45 feet long, averaging a driven length of 32 feet." "They were not less than 7 inches in diameter at the smaller end, and 12 to 18 inches (on an average, 14 inches) in diameter at the larger end. The trial piles averaged 12 inches in diameter in the middle. The heads of the piles were protected." The piles "were driven in rows $2\frac{1}{2}$ feet apart, and at transverse distances of 3 feet, all from center to center." But "intermediate piles, of very tough second growth oak, were frequently driven." "The piling machines were strongly and accurately made, with the ways bound with smooth plates of iron." They gave about one blow per minute. A Nasmyth hammer was used also, and gave sixty blows per minute. I have assumed that some of the piles tested were driven by it, although this is not stated in the reports. The author applies the test to a penetration of 0, but "the average distance driven by the last five blows was 1 inch." We therefore take the experiments as embracing $s = 0$, and $s = 0.2$.

As the result of the tests it was believed that for a pile driven 33 feet into the earth to the point of ultimate resistance with a ram weighing 1 ton (2 240 pounds), and falling 30 feet at the last blow, the

extreme supporting power due to the frictional surface, was 100 tons (224 000 pounds), or 1 ton per superficial foot of the area of its circumference.

This case illustrates the criticism already made of Mr. Crowell's formula that, like Major Sanders', it gives excessive loads where the penetration is very small. For the ordinary machine, our formula here gives better results than Mr. Wellington's, but for the Nasmyth machine, which I have assumed to have been used on the piles tested, his results are almost exact, while ours are double the actual load.

CASE 9.—Dordrecht, Holland, 1872. Bridge over the Maas. Temporary piles under staging. Procs. Inst. C. E., Vol. XLII, pages 213, etc. The author informs us that "the staging sank fully three-fourths of an inch, and during their erection the girders had constantly to be wedged up"; but leaves it to the ingenuity of the reader to deduce the other data required for our present purpose. The material seems to have been sand, with some mud; average depth of water 18 feet 9 inches; maximum, 22 feet 8 inches. The piles seem to have been driven either by a steam pile-driver delivering sixty blows per minute with a ram of 2 205 pounds falling 30 inches, or by "an ordinary hand engine"; ram, 992 pounds; fall, 6 feet 7 inches. Both cases are given in the table. However this may be, we are told that "some of them went in much too easily, being driven three-fourths of an inch and $1\frac{1}{8}$ inch with the last blows." But "it was arranged, where the penetration was more than three-eighths of an inch on the average of the last one hundred blows, to put in" additional piles. I therefore take three-eighths of an inch as the probable penetration. The load seems to have been about 13 440 pounds per pile.

CASE 10.—Aquia Creek, Case (*b*). See Case 1.

A pile 40 feet long, after sinking some 30 feet with its own weight and that of a 2 000-pound hammer, was given a blow of 2-foot fall, after which it sank $6\frac{1}{4}$ inches further in one minute under its own weight and that of the hammer, and then stopped. Four weeks later a 5-foot blow failed to move it, and a blow of 14 feet drove it only $4\frac{1}{2}$ inches. This case and the following one illustrate strikingly the effect of time in increasing the stability of a driven pile. In both cases the blow recorded was delivered four weeks after the settlement under the load. A load applied after the blow, as usual, would undoubtedly have made a better showing for the formulas.

CASE 11.—Aquia Creek, Case (*c*). See Case 1.

A pile 43 feet long received two blows of 2 feet each, and then settled under the hammer $1\frac{1}{2}$ inches in two minutes. Four weeks later it penetrated as stated in the table.

CASE 12.—Lake Pontchartrain Trestle, La., 1882. Letter from Mr. G. Bouscaren, June 25th, 1892. Facts recited by Mr. Bouscaren at Convention. About 6 miles of the trestle crossed the lake proper, and

the remainder (16 miles) crossed the adjoining sea-swamp. Four-pile bents, 15 feet between centers. Material of swamp, several feet of soft, black, vegetable mould, lying upon soft clay, with occasional strata, 1 to 2 feet thick, of sand. Piles sank from 5 to 8 feet of their own weight, and then about as much more with hammer (about 2 500 pounds) resting on head of pile. Two piles 65 feet long were driven, one on top of the other, and penetrated 9 inches with over 100 feet driven; but a 30-foot fall, thirty minutes after driving a pile, gave only three inches penetration. Piles, 65 to 75 feet long. Weight w of ram, about 2 500 pounds. Height h of fall, about 30 feet. Penetration, 3 to 12 inches. I have taken $s = 3$, 6, 9 and 12. "No settlement has been observed in the entire length of the structure to date." As there were four piles in a bent, and the bents were 15 feet apart, the load on each pile probably has not exceeded 22 400 pounds. Although the extreme load is not given, we include this case as forming an interesting contrast with Case 3, also given by Mr. Bouscaren.

Case 13.—Small experimental piles. "Resistance of Piles to be Ice-drawn," by John William James. Procs. Inst. C. E., Vol. XLI, page 191. Material of various kinds, chiefly compact sandy clay, compact gravelly clay, with surface soil, sometimes wet, sometimes dry. The "piles" were small sticks of oak, as follows: Square, 1 to 2 inches side; cylindrical, 1 to 2 inches diameter; flat, $3\frac{1}{4}$ x 1 inch. They had points of different shapes. A large number of experiments are recorded, of which the means given in the table are fairly representative.

Mr. Crowell's formula is hardly applicable to a case like this, for we have no way of finding his n, which is $\frac{1}{2}\sqrt{s'}$, where s' is the penetration as it would be under a blow of a 2 000-pound ram falling 20 feet or other equivalent (?) blow of 40 000 foot-pounds. Since none of the loads reach 2 000 pounds, it is plain that the mere weight of a 2 000-pound ram would have driven the pile indefinitely, and we can hardly doubt that any blow of 40 000 foot-pounds would have done the same and would also have abruptly terminated the experiment.

Case 14.—Ice-drawn pile. Observations made January, February, March, 1873. From the paper quoted in Case 13. Material not stated. Pile about 12 inches diameter, under a bridge constructed during the previous year, across a river confined by a milldam. The height h of fall was approximately 20 feet. "The driver was worked on the ice, and equal lengths of the piles did not project above its surface when the last blow was delivered." The penetration s is said to have been one-half inch. The load was estimated by the uplifting force of ice. "The piles held their ground, and the ice broke away, until it had attained a thickness of 15 to 16 inches, when the piles began to come up." The lifting force was estimated, from experiments by the author upon small, smooth sticks, at 18 850 pounds per pile. Deducting weight of superstructure, about 3 000 pounds, leaves 15 850 pounds as the estimated force required to draw the pile.

In view of the smallness of the actual load in this case, it may reasonably be asked whether much of the energy of the blows was not consumed in overcoming the friction of the piles against the ice through which they were driven. It will also be noticed that the force was an up-lifting one, and that its amount was estimated by means of the adhesion of ice, as derived from experiments with small, smooth sticks. The weight of ram, height of fall and penetration, were all taken by the author of the paper at hearsay, from a man who had worked on the bridge. The case is, in fact, one of exceptional doubtfulness in all respects.

CASE 15.—Proposed Cambria Reservoir, Philadelphia, 1883. Notes by the writer. Material, wet, decomposed mica schist. A wrought-iron pipe, 3¼ inches outside diameter, 3 inches inside, was inserted in a bore-hole 6 inches in diameter and 30 feet deep, and driven 14 feet to rock. The lower end of the pipe was dressed to an annular cutting edge.

LOAD.—After several hours' work with block-and-fall, the pipe was pulled in two by using two hydraulic jacks of unequal power, one on each side, by means of which the pipe had been raised 8 inches. The fracture took place in the thread, where the wall thickness was reduced from one-fourth to one-eighth-inch, leaving a cross sectional wall area of $3\frac{1}{8} \times 3.1416 \times \frac{1}{8} = 1.23$ square inches.

Since the pipe had risen slightly under the pull which soon after caused its rupture, the latter was evidently about equal to the resistance of the pipe to being withdrawn. We can estimate at the amount of this pull by estimating the tensile strength of the iron at the point of rupture. The conditions of the experiment were very crude, but considering that the pipe was no doubt weakened by canting from side to side under the unequal forces exerted by the two jacks, as well as by repeated blows in driving, and repeated tensile strains in drawing, and by the removal of the outer "shell" of the iron in cutting the thread, the tensile strength could hardly have exceeded 40 000 pounds per square inch, and, on the other hand, it was perhaps not less than 20 000. For the force required to pull the pipe asunder (which we assume to be equal to the upward resistance of the pile, or "extreme load"), we have, according to our estimate of the tensile strength :

20 000 pounds per square inch x 1.23 square inches =, say, 25 000 pounds, extreme load.

30 000 pounds per square inch x 1.23 square inches =, say, 37 000 pounds, extreme load.

40 000 pounds per square inch x 1.23 square inches =, say, 50 000 pounds, extreme load.

CASE 16.—Pensacola, Fla. Charles B. Stuart's "Naval Dry Docks of the United States." Also, a paper on the resistance of piles, by

Hamilton E. Towle, C. E.; Annual Reports of the American Institute, 1867–68, pp. 890, etc. General Delafield, in his Memoir on Foundations in compressible soils, pp. 18 and 20, quotes Stuart, and makes this appear as two cases.

MATERIAL.—Clean, hard, sharp, white quartz sand. All the sand would pass through a sieve having openings one-twelfth of an inch square. Water filtered through it came out perfectly clear. One cubic foot of it would hold 6 quarts of water. The 2-ton hammers could only drive about 20 feet. The water was about $11\frac{1}{2}$ feet deep.

Seven piles, selected as representing the average of all, were tested with upward pulls of 20 000 pounds each without moving, and one of these was afterward tested with upward pulls sufficient to cause motion (as recorded below) and finally withdrawn. This pile was 29 feet long, 16 feet in sand, including its point, 2 feet long. One foot of this length was in loose sand, which had been excavated and had fallen back. The average diameter of the part in the sand was $13\frac{1}{2}$ inches, including the bark. Weight of pile, 1 632 pounds. This pile was tested two months after it was driven.

Our two authors have refrained from giving us w, h or s for the trial pile. The following guesses at their values are the result of laborious study of their remarks respecting the foundation piles as a whole, and are to be taken as of very doubtful correctness.

1st.	$w = 2\,200$,	$h = 30$,	$s = 0.5$.
2d.	$w = 4\,100$,	$h = 33$,	$s = 0.0$.
3d.	$w = 4\,100$,	$h = 10$,	$s = 0.6$.

The tests on the trial pile resulted as follows:

78 000 pounds	....No movement,	
80 000 "	....Resisted $\frac{1}{2}$ minute, and then rose very slowly,	Rose $2\frac{1}{2}$ inches in 30 minutes.
82 000 "	....$1\frac{2}{3}$ minutes,	
83 000 "	... $\frac{1}{2}$ minute,	
60 000 "	....18 hours. No movement.	
64 000 "	..	Rose 3 inches in one hour, 6 inches in all.
74 000 "	..	
50 000 "	....For two days. No movement.	

The very small loads obtained by the test in this case seem to confirm the view already expressed that the resistance of a pile to an upward pull must be less than that to a downward pressure. This is especially noticeable in comparison with the Brooklyn tests, Case 8, where the conditions were nearly similar, but where the pile (tested by pressure) bore much greater loads. The sand at Pensacola was remarkably pure, and hence could probably exert little resistance to being broken up, while offering great resistance in the opposite direction, as is shown by Mr. Towle's statement that at a depth of 15 feet a 2-ton ram (falling 33 feet) rebounded nearly a foot.

CASE 17.—Albert Dock, Hull, England. Removal in January and February, 1880, of coffer-dam built in 1874. "Note on the Friction of Timber Piles in Clay," by Arthur Cameron Hurtzig, Procs. Institution of Civil Engineers, London, Vol. LXIV, Session 1880-81, Part II.

MATERIAL.—Compact bluish clay, above which there were from 3 to 5 feet of peat, and above this silt and sand in places. Piles of ordinary rough Memel bark timber, from 10 x 12 to 14 x 15 inches. Average, $12\frac{1}{2}$ inches square. From 20 to 40 feet long; average, 40 feet. Driven length, from 6 to 30 feet; average, $18\frac{1}{4}$ feet. Most of the piles were driven from 10 to 20 feet into the clay and were nearly or quite in that material alone; but a few of the shorter piles, driven in a sloping side of the dock, were entirely in the silt, while a few others entered the peat without reaching the clay. The piles were driven close together in two rows 5 feet apart, forming a coffer-dam, the space between the two rows having been filled with puddled clay "to above high water-mark." Before the piles were withdrawn the puddle was removed down to a level "rather under high water-mark of ordinary neap tides."

The height h of fall varied from 5 to 6 feet, and the penetration s from 0.5 to 0.75 inch. I have taken the extreme cases. Four hundred and twenty piles were withdrawn and 300 observations recorded. The force was applied by men working a winch and estimated by testing that of the men in lifting known weights. The piles were drawn consecutively, so that one side of each pile was nearly or quite free from frictional contact, the opposite one was in loose contact with the adjoining pile, and only the remaining two sides resisted by friction with the ground.

The average total force required to draw a pile was 75 869 pounds. The author deducts from this 2 340 pounds (= 12 x 12 inches x 15 pounds per square inch) for suction, and 2 240 pounds for the weight of the pile, leaving, say, 71 300 pounds as the frictional resistance to drawing the pile.

J. FOSTER CROWELL, M. Am. Soc. C. E.—I want to reply briefly to one or two points which Mr. Wellington has made. So far as the mathematical demonstration, which (referring to the figures on the blackboard) he says, is the only satisfactory one that can be made is concerned, I have nothing at present to say. The object of the paper is to present clearly what the usual complications in practice are and to suggest a way out of them to a more simple treatment. Mr. Wellington has not read my paper very carefully or he would have seen that instead of advocating small penetrations I expressly recognized the contrary principle and stated that too much importance was placed on small sets. I agree with him in thinking that an ordinary penetration is proper. I agree with him that penetrations of much less than 1 inch come nearer to results that can be treated with suspicion, and I have stated that the only reference to the smaller penetration in the table was to show the theoretical limit to the application of the modi-

fied formula in the extreme case. I stated that I did not believe in considering very small penetrations.

There is one other point which I do not think Mr. Wellington traverses either. I am very glad to know by the way that Mr. Wellington is the author of the *Engineering News* formula and I think my paper is a support of that formula; but in using it I find that where conditions are very favorable his formula gives lower results comparatively than when they are not; when everything is very near right the results are somewhat lower than I feel justified in adhering to. His statement that the formula with the constant 1 expresses the extreme condition is, I think, only correct when applied to piles of a certain size. The reason for varying that constant 1 is to make it applicable to different conditions; I will admit that this is a refinement that need not always be gone into. But if we are not going to be governed by an absolute empirical value of the constant in all cases, a means of varying that value to suit the exigencies or necessities that engineers will discover when driving piles, seems to me a desirable end. I have frankly confessed I was compelled to adopt Mr. Wellington's formula and only suggested how it could be improved. I endorse practically everything Mr. Wellington has said about unnecessary refinements. The operation of driving a pile is a very crude one at the very best. All this investigation applies more to the stratum underneath the pile than to the pile itself. The pile itself is an indication of what is going on below. In practical work I have gone to the expense of driving very long piles way below the piles I was going to use, in order to ascertain the conditions, and have used the same general principle of the standard blow as in this formula.

I have only a word or two to say in regard to Mr. Trautwine's discussion; first, to thank him for the care and thought he has expended in getting together these statistics to help throw light on the general subject; and, secondly, to relieve his mind of the fear that seems to exist, as to the correctness or the thoroughness of the comparison in diagram No. 2, of the two curves of the Trautwine formula. Those curves exactly represent two conditions of the formula; the first, the formula as it appears in the first edition of Trautwine's "Engineers' Pocket Book," which appeared in 1872, and of which, I believe, I am the possessor of the first copy given to the public. In that originally—it was the same, I think, in one or two editions—the formula is in this shape:

$$\text{Extreme load in tons,} = \frac{\text{Cube root of fall in feet} \times \text{Weight of hammer in tons,}}{\text{Set in inches} + 1.} \times 60.$$

If the weight of hammer be given in pounds, the coefficient becomes $\frac{60}{2\,240} = .0268$ and we have L (in tons) $= \frac{\sqrt[3]{h} \times .0268}{s + 1.}$ as in some editions.

In the later editions the formula has been changed only in one respect—the decimal has been changed to .023. The maximum load represented on the first formula is one-half, while on the second it is one-third. In preparing the diagram, I took the modern edition, but it occurred to me that perhaps some of the members, in examining the papers, would refer to their earlier editions of the book in which the other value would be given, so I plotted the two curves to represent the two conditions.

Mr. Trautwine has said that it was difficult to ascertain the conditions in regard to those piles that were driven at the Brooklyn Dry Dock because he had some difficulty in admitting "n and n_1." My modification is not intended to be retroactive.

In reference to one other point where in stating the view, in which I think he agrees with me, that with his formula a low fall gives excessive values, he intimates that in most cases there would not be very low falls. I will state that the average stroke of a Nasmyth steam hammer in ordinary use would be from 2½ to 3½ feet; in ordinary circumstances, the weight of ram is, say, 5 500 pounds. If we take this weight with the full stroke 40 inches, there would be developed a blow of 18 315 nominal foot-pounds; now, if we were to take a hammer of 2 000 pounds and divide this value by the 2 000 pounds we would find that we would require a fall of $9\frac{15}{100}$ feet to produce the same blow, neglecting resistances in each case. Now, applying the Trautwine formula (later form) in the first case, you will find we obtain 70 368 pounds as maximum load, but in the second case only 35 838 pounds, or just about one-half; that shows that this objection to the cube root is not a theoretical objection when the steam hammer is used, and I think those who have occasion to drive many piles will make more use of the steam hammer in the future than in the past.

CHARLES B. BRUSH, M. Am. Soc. C. E.—Mr. Chairman, the discussion that we have had this morning has been interesting and suggestive. It has been valuable because it is suggestive, but we ought to consider that it has been purely theoretical. It is based on the assumption that there is a comparative uniformity in the soil which we are penetrating, and the conditions under which we are operating. If there is a lack of uniformity, the whole theory falls to the ground.

There are two things to be considered in driving piles: First, that the pile is driven home; second, that after it is driven home, it is not broomed. The danger is in continuing to drive after a pile is thoroughly bedded, and thus brooming the pile. This is not theoretical, it is a practical danger. I have known a number of instances affecting large buildings where serious results have followed. The danger consists in the fact that often we do not apply our common sense, but do apply a formula to a condition which is continually changing, and the result is that we drive our piles too hard, or not hard enough. Our

discussion has been pursued on the basis that we are driving through uniform material.

It seems to me that there is only one safe way to proceed where we have to pile the foundations of the class of important buildings, such as are enumerated on page 112, to which this paper and this discussion especially refer. Borings should first be taken to determine the character of the material to be penetrated. We should also rely on the indications as the work proceeds. I do not think it is safe to rely on formulas. Experienced men should be employed, and if the borings showing the strata to be penetrated are furnished to them, the pile can be driven with perfect safety. A penetration of less than half an inch indicates the probability that the pile is driven home.

Mr. WELLINGTON.—Mr. Brush's argument is a good one from his premises, but it would be stronger and more valuable if his premises had been right. He assumed that this has been, so far, a theoretical discussion of the formulas. In a sense this is true, but what are the formulas? They are simply the closest possible application of known physical laws to the known facts of experience. They simply say that when you have driven a pile into the ground and observed its rate of penetration under given blows, you know all that you can know about the resistance of that particular soil to that particular pile, and that it remains only to determine how large a proportion of the resistance observed you are warranted in placing upon it as a safe, permanent load. That part of the question is necessarily theoretical, but its basis is the eminently practical observations on each successive pile. Mr. Brush proposes as a substitute to such reasoning from actual observations of set, to start in beforehand and put down a test pit, and from the general look of this pit to guess how much his piles will bear after he has driven them. In most cases it is utterly impracticable to put down test pits in soil demanding piling to a depth considerably below the probable depth of the piles, which is what must be done to have the test pits of any use; and even supposing them sunk, I do not see that he has got anything but simply guessing after all. Each pile as it is driven, if properly observed, makes for itself the best possible test pit for the end in view. The great fact which you need to know, and the only fact which materially aids the judgment is: what was the resistance to penetration of the pile when driven home, and was this penetration uniform or not? The only thing remaining to be determined is: what proportion of that resistance you can trust to in practice? That is the fact we have been discussing, necessarily from a theoretical standpoint.

G. BOUSCAREN, M. Am. Soc. C. E.—I think the trouble about applying a formula to pile-driving is that, in doing so, we gauge the resistance of the soil from the penetration of the pile while it is being driven. It often occurs that the resistance of the soil is greater when

the pile is being driven than it is after the pile is driven, and *vice versa*. Here are some examples which occurred in my practice.

In driving piles for trestles for railroads through Illinois, I had adopted the rule to stop the pile-driving at the penetration of 1 inch; the hammer weighed from 1 800 to 2 200 pounds, the fall was about 25 feet. I generally found the rule to work very well.

When I came to build a railroad in Louisiana through the swamps of Lake Pontchartrain, I found that the penetration of a 65-foot pile under the last blow of a 2 500-pound hammer, falling 25 or 30 feet, was from 9 inches to 12 inches. Putting two of these piles, one on top of the other, and having a total penetration of 120 feet, the penetration under the last blow was still 9 inches. I thought I would see what the penetration would be if I gave the pile a rest, and I stopped the driving about twenty-five minutes after the pile had gone in about 40 feet; the first blow of the hammer, when the driving was resumed, gave a penetration of 1½ inches. I concluded in this case not to consider the penetration under the last blow, but to use piles from 60 to 70 feet long. About 22 miles of trestle-work were built that way, and none of the piles ever settled.

Here is a case which is the opposite. I built a pile trestle across what was then called the "Slough," back of Bloody Island, in East St. Louis. It was built with three pile bents, the penetration there was from 2½ to 3 inches, less than one-third of what it was in the case of the Lake Pontchartrain trestle, and within two weeks after the bridge was opened, we could not get a train over it; it had settled irregularly from 1 to 2 feet. That was in 1867, when locomotives were much lighter than they are now.

It is often impossible to tell, without actual experience, what reliance can be placed on the resistance of the soil as evidenced by the penetration of the pile.

C. Wheeler Durham, M. Am. Soc. C. E.—In 1870 I was engineer for the Chicago and Northwestern road, in charge of the extension of the line from Madison to the Mississippi River, at La Crosse. One part of the line crossed a marsh in which there was a stream of perhaps 30 or 40 feet in width, requiring about 800 feet of bridging. The piles were ordered 45 feet in length. When we came to drive them, the piles went down to within a foot of the surface without a blow of the hammer at all, they were simply pushed down by the hammer resting upon them. We saw that these 45-foot piles would not answer the purpose, so we stopped operations to get a new lot of longer piles. I forget exactly the circumstances which caused us to try these piles a second time in a day or two, but we found then that they did not budge under a fall from the full drop of the hammer; it would not stir them at all. So we pushed the balance of those piles down in the marsh for the 800 feet of distance, capped them and built trestle-work upon them, and I

suppose they are carrying the Chicago and Northwestern road to this day.

L. M. HAUPT, M. Am. Soc. C. E.—I think that it might be of interest to the members to know how piles were driven in this vicinity even without a hammer. On the Eastern Shore it is the practice in building small wharves for the fruit trade and light-draft steamers, to take the trees of the neighboring forests, and, after erecting a temporary staging with a projecting cantilever upon which two darkies can stand, a pile is placed between them and is rocked to and fro, its toe resting on the bottom, until by its weight and the swaying motion, it beds itself to the required depth. Any cessation of the movement causes the sand to settle and pack around the pile so quickly that it is impossible for the men to start it again. In this way the work is done very cheaply and efficiently with an inexpensive plant. In the cases cited by Mr. Bouscaren, I think, possibly, the firmness of the piles in the Louisiana swamps is due to an under stratum of sand which settles around and clamps them in place. In the other case mentioned, back of St. Louis, there must have been some mud which was easily moved by a current. I should be pleased to know more about the sections through which they were driven.

WILLIAM P. CRAIGHILL, M. Am. Soc. C. E.—I have just been informed of a peculiar case of pile-driving that occurred in some work in charge of a friend of mine who was improving the Caloosahatchee River in Florida, where he had to drive piles and had no pile-driver; the piles also were of a kind that would not last long under the blow of a ram. The situation was peculiar; he disposed of it in the following way, and I would be glad if the gentleman would give attention because I want to get a formula which will apply to cases of that sort. Fortunately, he had a very fat man in his party, so that after placing the pile on end, he put this fat man on top of the pile; this gave the first motion to the pile. In order to keep up the interest in the subject he got two or three men to shake the pile vigorously and try to shake the man off. That was the amusing part of the operation; they shook the pile and the fat man clung on. Fortunately, there was a water-jet available, so that with all these contrivances the piles were carried down to the rock, but I should like to know in the next edition of the formula the form it would take to cover that case.

W. M. BLACK, M. Am. Soc. C. E.—I cannot tell you very much about the bearing qualities of piles, but I would like to invite the attention of the Society to the lack of formulas and data regarding the safe load for piles sunk by the water-jet. This method of sinking piles is coming more and more into use. It has been used for many years by the United States Government Engineers in their work, and is now used quite generally in sandy soils all over the country.

Some years ago iron screw piles were used in the Government Pier

at Lewes, Delaware. In attempting to screw them down into position the friction was so great that several were broken by torsion. A water jet was then applied to the top surface of the screw blades and the sinking was successfully accomplished.

The "live-load method" of pile-driving in the Caloosahatchee River, as described by Colonel Craighill, proved very effective. From 150 to 170 piles were sunk daily to a depth varying from 4 to 6 feet in the sand, by the aid of a water-jet. These could not have been driven by the ordinary method, even had the necessary plant been available.

In southern waters special precautions are necessary with pile-work to avoid the ravages of the teredo. One plan is to use wood which they attack but little. In Florida, the palmetto is extensively used for this reason. As is well known, this wood has a hard shell and soft interior, like a cornstalk, it will not bear a heavy blow from a hammer. In one case palmetto piles had to be sunk to a depth of from 12 to 15 feet through hard sand. This was done successfully by the water-jet aided by light taps from a hammer. A slight protection was given to the point of the piles by sheet iron. The pile to be sunk was raised to position and the hammer placed on it. The water-jet was then applied. When the descent of the pile became too slow, a light blow was given by a hammer fall of 3 to 6 inches. The pile was given a slight motion of vibration and the pile and the water pipes were kept moving constantly to prevent the sand from closing in and holding them. This seems to me an important point in pile-driving which has not been much touched on in this discussion—the necessity for keeping the pile moving.

In using the water-jet, we found it necessary to have the quantity of water ample. In fact, we found the essential to be quantity rather than velocity. The velocity had to be sufficient to cause the water to penetrate the sand below the foot of the pile and make it "live" or "quick," and the quantity so great that the water had to escape by rising along the skin of the pile, preventing surface friction.

In the surveys of the harbors of St. Augustine and Key West, borings had to be made through the sand, to depths varying from 12 to 30 feet. This was done by means of the water-jet, and the method adopted illustrates its action. In compact sand a 2-inch pipe was coupled to the discharge-pipe of a hand-pump and forced down to a depth of from 5 to 10 feet. When it was stopped by the sand closing in on it, a 1-inch pipe was coupled in its place, lowered through the larger pipe and forced down to the required depth. The water rose along the surface of the small pipe and through the annular space between the two pipes, bringing with it samples of the strata of sand penetrated. Thin strata of shell gravel were also bored through by this apparatus.

CHARLES WARREN HUNT, M. Am. Soc. C. E.—At the pier at Lewes, Delaware, of which Captain Black has spoken, the piles were of solid wrought iron, from 22 to 45 feet in length, and from $5\frac{1}{2}$ to 8 inches in diameter, with cast-iron screws $2\frac{1}{2}$ feet in diameter. When the work was in an experimental stage, the attempt was made to screw down the piles into compact sand with the aid of a jet of water forced between the flanges of the screws to the under side of the flanges. This was not successful, several of the piles being broken by torsion. An inspection of the screws developed the fact that the greatest friction was on the upper surface of the flanges, and it was found that, when the jet was applied to the latter, no more difficulty was experienced, and all the piles in the pier were placed in that way.

The gain by the use of the water-jet on these screw-piles was measured mechanically, and was stated in the Report of the Chief of Engineers,* as follows: "That $\frac{9}{10}$ths of the total pressure on the rope which connects the power with the resistance at once disappears, in effect, on the application of the water-jet."

ROBERT B. STANTON, M. Am. Soc. C. E.—As shown by the discussion this morning of Mr. Crowell's paper, it seems to me the principal value of this investigation is, to arrive at some method of establishing the safe load we can put upon piles, after they have been driven, not only into, but in some cases through, various kinds of material.

I shall not take up your time in giving you any of my experiences in pile-driving, although I have had considerable in the Western States; but I only want to make a suggestion for the continuation of this investigation, and the extension of the formulas that have been given in the paper. Every one seems to have treated the pile as a column, resting upon a foundation, which will be reached by this method of driving with a heavy hammer, and the formulas are all made with this as a basis.

Now, from my experience, and such experiences as have been related by Mr. Durham, Mr. Trautwine, and others, what we need is some means of ascertaining the safe load that may be placed upon piles that have been driven into certain materials that hold the pile and the structure upon it, by the pressure or grip upon its sides, exerted by the material, not during the blows of the hammer, but more or less shortly afterwards. This "grip" is not simply friction; it is something more. The end of the pile is practically through the material, and rests upon nothing. It is, then, not a column. Cannot Mr. Trautwine put together the data he has gathered, and give us some light upon this point?

HENRY B. SEAMAN, M. Am. Soc. C. E.—With reference to the use of concrete filling, to avoid corrosion in cast-iron piles, I would ask if any one present has known cast iron, with its original sandy surface, to corrode in sea water.

* Report of Chief of Engineers, 1873, p. 860.

Captain BLACK.—I have seen instances of cast iron corroding in salt water, though with white cast iron the corrosion is very slow.

L. M. HAUPT, M. Am. Soc. C. E.—In this discussion there has been no reference made to a very recent experience in sinking caissons as well as piles by pneumatic process. I rise to call attention to the experience of Mr. J. P. Nelson in sinking caissons at one of the inlets of the Gulf coast by means of the pneumatic process. There are a number of nozzles arranged around hoops which are not quite large enough to fill the interior of the caisson. These are placed inside the cylindrical caissons which are open at top and bottom. In fact, they are merely large sections of wooden tubing 16 feet long and about 8 feet in diameter which are placed vertically and in contact. The jet is applied to each in succession and it is rapidly sunk to the required depth. By this process from 75 to 80 linear feet of revetment is built in a day. The caissons are then filled to the top with sand and will be covered with rip-rap as rapidly as it can be obtained. The two parts of the structure are designed for mutual protection, the one, from *teredo;* the other, from waves.

WILLIAM WATSON, M. Am. Soc. C. E.—That mode of sinking piles and masonry curbs was very extensively practiced by the engineers of Calais and also in crossing the Marmande in the south of France. It is hardly worth while to enter into the details of the subject at present, but it will be found very fully discussed in the forthcoming United States Report on the Paris Exposition, which is already printed, but not yet ready for distribution.

FRANK W. SKINNER, M. Am. Soc. C. E.—There is one point for consideration in the sinking of piles by jets that has not been touched upon that is perhaps not very important, but is of some particular use occasionally ; that is, the sinking of piles in a limited headroom. An instance in point was the recent removal of the Hotel Brighton at Coney Island. It was in great danger of being destroyed by the encroachments of the sea in winter storms. The building, 700 to 1 000 feet front, was moved up as a whole several hundred feet back from the water, and then replaced on pile foundations ; there was, practically speaking, no headroom at all to set those new piles, and it was accomplished very successfully and rapidly by the jet system which enabled them to form a cavity for the pile to be rocked and twisted into position in.

I knew of another instance in connection with the building of the Jumping Point Bridge at Seabright, N. J. The piles had been driven before my arrival there so that I did not see the exact operation, but as nearly as I could ascertain afterwards they were intended to be sunk by pumping water through the interior of a hollow cast-iron pile and allowing it to escape through the conical screw disc of the pile which had perforations in the center. It was not very successful and

a good many piles were driven down with a large heavy ram; after they went down a few feet the bottom was found hard and caked and it was very difficult to get a grip on the sand, which broke up in chunks. After the bridge was constructed a good many of the piles had to be replaced, both through breakage in driving and through distortion and displacement afterwards.

Captain BLACK.—Was that in a case of water-jet used in gravel? I do not believe that the water-jet acts by scouring away the sand at the bottom of a pile, but by making it quick, and by taking away the skin friction on the pile. It is essential that the sand should be fine enough and close enough to make the line of least resistance for the escaping water along the skin of the pile.

Mr. SKINNER.—My recollection is that the pile did not reach the gravel bed.

F. P. DAVIS, M. Am. Soc. C. E.—The Captain says that he does not think that the water causes a scouring out under the pile. I had occasion to drive some piles through very hard, compact sand where it would take 200 blows of the hammer, falling from 20 to 30 feet to drive the piles 20 feet. I have seen the piles stick, the water still being applied. By giving a light blow with the hammer the pile would take a sudden start and go from 4 to 5 feet, showing that the sand had been scoured out. I am satisfied the water-jet will scour the sand to quite a depth below the bottoms of the piles.

BENJAMIN REECE, M. Am. Soc. C. E.—This discussion affords a marked illustration of the frequent contentions which arise from a failure to recognize, or a determination to ignore, the other side of a proposition. Thus, upon the one side an effort is made to establish a formula by which the penetration of a pile under a falling ram affords the data for a mathematical determination of its value in the support of given loads. Upon the other side appears a very evident attempt to discredit any formula, without reference to its scientific accuracy, and a number of illustrations are presented to show that the application of any formula would be worthless. The fallacy leading to such extreme differences of opinion rests upon the failure to recognize the element of time as being an essential factor in the conditions to be determined.

That the bearing capacity of a pile bears some relation to its penetration under the weight and fall of the ram but few persons would absolutely dispute, but, manifestly, the conditions which exist after the pile is in service are entirely different from those which obtain while the pile is being driven. Thus, it is noted some piles which penetrated several inches to the blow while being driven, resisted the same weight and fall without any perceptible penetration after the pile had been permitted to rest, thus affording a restored equilibrium to the material through which the pile had been driven. Upon the other hand, piles which were driven home with difficulty, have settled down

under seemingly inadequate loads, which would indicate that the material failed to find adherence to the pile, thus permitting the percolation of water tending to soften the material which sustained the pile.

It is evident then, that if the penetration of the pile, after a succession of blows, is taken for a basis of calculation as to what the pile will support in service, no formula, no matter how accurate, can be relied upon; for the last blow of the ram, in the process of driving, occurs when the material surrounding the pile has been disturbed in the act of driving, whereas the effects of such disturbances have generally disappeared by the time the piles are loaded. Manifestly, the bearing capacity must be determined from data corresponding to conditions of service. This at once suggests that such tests should be made by dropping the ram on the driven pile after affording it some period of rest. Thus it would be comparatively easy before moving the driver, to test on Monday morning the pile last driven Saturday evening, and the penetration of the pile observed. The penetration due to the last blow each afternoon could always be compared with that resulting from the first blow on the mornings following, and, if deemed necessary, other tests could be made after longer intervals of rest. But little time would be lost in following this practice, and at the same time the data so obtained could be used with no small degree of accuracy to determine by means of formula the bearing capacity of piles in service.

G. B. Nicholson, M. Am. Soc. C. E.—Mr. Crowell, in his conclusion, expresses the hope that the discussion may call out practical examples taken from actual experience of the behavior of piles in service. In answer to this wish, the following are submitted:

In the year 1883, I set up a turntable on a pile foundation in the lower part of the City of New Orleans, 2 300 feet from the bank of the Mississippi River, for the New Orleans and North Eastern Railroad. As is generally known, the soil of that region is a compressible, wet alluvium, making the use of piles for the support of the pedestal of a turntable the best form of foundation.

The turntable I allude to was of the ordinary wrought-iron type, and 54 feet long. The pedestal was supported on a foundation of six yellow-pine piles, freed from bark, 10 inches in diameter at the small end and 14 inches at the large end. The piles were capped with a grillage of two tiers of 12 x 14-inch creosoted yellow pine, and secured to the piles and to each other by 1-inch drift-bolts. The pedestal of the turntable was bolted to the top tier of timber. Experience in driving a large number of piles in the vicinity a short time previously had given me a great confidence in the sustaining power of piles having a rather large penetration or sinking at the last blow of the ram.

The record of the pile-driving is as follows:

No. of pile.	Length after framing. Feet.	Sinking at last blow. Inches.	Fall of ram, last blow.	No. of blows.
1	35.5	12	35	30
2	34.5	12	30	20
3	27.5	12	35	30
4	30.	9½	30	29
5	29.5	18	35	27
6	27.5	9½	32	25

Weight of ram, 2 825 pounds; weight of turntable complete, 32 400 pounds; weight of grillage on top of piles, 3 700 pounds; weight of engines, with tenders, using the turntable, from 116 000 to 156 000 pounds—50 per cent. at present are 155 000 pounds; approximate average daily use of turntable by engines and tenders since building, thirteen times. The turntable has been in existence on its original foundation, without repairs, for nine years, and no appreciable settlement has been discovered.

It is a well-known fact that if a pile is allowed to rest after being partially driven, and the driving resumed after a considerable interval, that the first penetration in the new driving will be considerably less than the last penetration when the driving was stopped. I do not know what the ratio is in dry soils; but I have several illustrations for pile-driving in the wet alluvium about New Orleans as follows:

Weight of ram. Pounds.	Fall of ram. Feet.	Penetration before resting. Inches.	Interval of resting. Hours.	Record of driving after resting. Penetration in inches, fall of ram in feet
2 750	27	7¼	48	$\frac{0}{17}$ $\frac{1}{17}$ $\frac{1}{17}$ $\frac{1}{17}$ $\frac{1}{17}$ $\frac{1}{17}$ $\frac{1}{17}$
3 000	35	6	24	$\frac{1}{35}$ $\frac{2}{35}$ $\frac{3}{35}$ $\frac{5}{35}$ $\frac{5}{35}$ $\frac{5}{35}$ $\frac{5}{35}$ $\frac{5}{35}$ $\frac{5}{35}$ $\frac{5}{35}$ $\frac{7}{35}$ $\frac{7}{35}$
3 000	20	6	12	$\frac{1\frac{1}{2}}{15}$ $\frac{2\frac{1}{2}}{20}$ $\frac{2\frac{1}{2}}{20}$ $\frac{3}{20}$ $\frac{3\frac{1}{2}}{20}$ $\frac{3\frac{1}{2}}{20}$ $\frac{5}{25}$ $\frac{5}{25}$ $\frac{3\frac{1}{2}}{20}$ $\frac{3\frac{1}{2}}{20}$
3 000	18	7	1	$\frac{6}{35}$ $\frac{5\frac{1}{2}}{30}$ $\frac{3\frac{1}{2}}{25}$ $\frac{6}{22}$ $\frac{6}{22}$ $\frac{6}{22}$
3 350	18	7	48	$\frac{1\frac{1}{2}}{20}$ $\frac{2\frac{1}{2}}{20}$ $\frac{2\frac{1}{2}}{20}$

NOTE.—The expressions in form of fractions mean that upper figures represent penetrations of pile in inches, and lower figures the fall of ram in feet.

It may not be out of place here to call attention to erroneous results obtained in pile-driving according to a method common in the Southwest—that is, the practice of never detaching the ram from the line. Two or three turns of the line are made on the drum of the winch, and when the desired height is reached, the line is slacked on the drum and the ram falls, carrying the line with it.

The braking power of the pulley at top of pile-driver frame and on the drum of the winch, and the consequent loss of momentum of the ram is illustrated by the following experiments:

First.—A pile penetrating $\frac{5}{10}$ feet, with a 40-foot fall of a 2 470 pounds ram, with the line attached to ram and slacked on the drum, penetrated $\frac{7}{10}$ feet with a 40-foot fall when the ram was allowed a free fall by cutting the line at the top of the leads.

Second.—A pile penetrating $\frac{7}{10}$ feet, with 45-foot fall of 2 750-pound ram, with line attached to ram and slacked on the drum, penetrated $\frac{9}{10}$ feet when the ram was allowed a free fall by cutting the line.

Third.—A pile penetrating $\frac{32}{100}$ feet, with 46-foot fall of 2 500-pound ram, with line attached to ram and slacked on drum, penetrated $\frac{4}{10}$ feet when ram was allowed a free fall by cutting the line.

The replies to this discussion will be published in a subsequent number of the *Transactions*.

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NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

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THE CANTILEVER HIGHWAY BRIDGE AT CINCINNATI.

By GUSTAVE KAUFMAN, M. Am. Soc. C. E., and
F. C. OSBORN, M. Am. Soc. C. E.

GENERAL DESCRIPTION OF THE WORK.

By GUSTAVE KAUFMAN, M. Am. Soc. C. E.

During the years 1890 and 1891 the Cantilever Highway Bridge described in this article was built across the Ohio River, between the cities of Cincinnati, O., and Newport, Ky. The terminus in Cincinnati is at the corner of 2d Street and Broadway, and in Newport it is at the corner of York and 3d Streets. The roadway of the bridge is 24 feet wide in the clear, with two sidewalks each 7 feet wide. The total length of the structure is 2 966 feet. The main engineering feature is the cantilever span, 520 feet from center to center of piers.

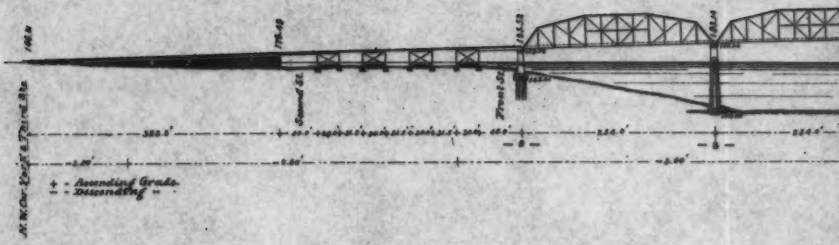
The bridge is located between the Louisville and Nashville Railroad Bridge and the Old Cincinnati Suspension Bridge, and but a short

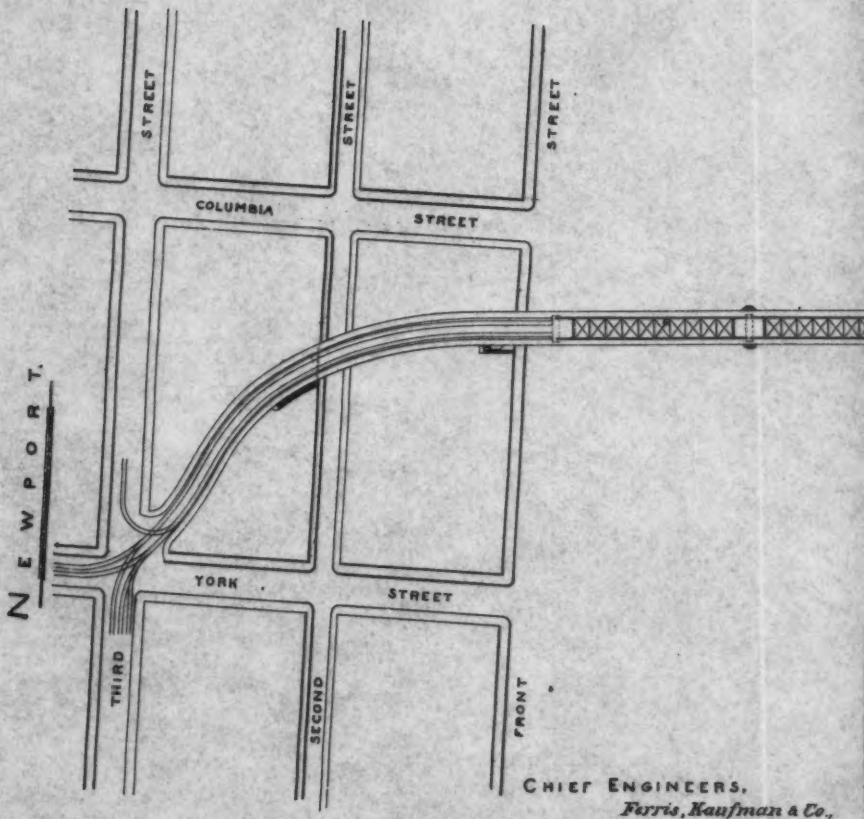
distance above the mouth of the Licking River. Prior to the construction of this bridge, the highway traffic between the two cities was principally accommodated by a ferry company with two large ferry boats. The Louisville and Nashville Bridge, which is supplied with very narrow roadways and sidewalks, accommodated the street-car traffic and a portion of the other highway traffic, but was not popular on account of its location and the interruption of highway traffic during the passage of trains.

The site of the new bridge is very favorable for economical construction, from the fact that a peculiar limestone formation extends across the river at this point. The top of this formation is an irregular triangle in shape, the base of which is on the Kentucky side and the apex on the Cincinnati side. On the Kentucky side at extreme low water this formation is exposed; the base of the triangle is about 1 400 feet long, and extends from the mouth of the Licking River to a point midway between the bridge under discussion and the Louisville and Nashville Bridge. The top of the formation maintains the level of extreme low water about two-thirds the distance across the river, where it drops suddenly, and for the balance of the distance across the river the top of the rock is from 5 to 7 feet below low water. The sites of other bridges built across the Ohio River at Cincinnati were by no means so favorable, and it was necessary to go to considerable depths to obtain suitable foundations for their piers; notably the Chesapeake and Ohio Railway Bridge, where rock was found about 52 feet below low water.

For many years the fear of insufficient revenue prevented the construction of a highway bridge at this place; but the development of the electric street railway and the necessity for rapid transit between the cities of Cincinnati and Newport furnished the requisite impetus and led to the formation of the Central Railway and Bridge Company for the purpose of constructing this bridge. The preliminary surveys and the general lay-out were prepared by G. Bouscaren, M. Am. Soc. C. E., in the early part of 1887. The general plans and the location of the channel span were approved by the Government in April, 1888, and authority at the same time was given for the construction of the bridge.

The general elevation and plan of the bridge; the gradients, lengths of spans and height above water are shown on Plate XXVII. A gen-

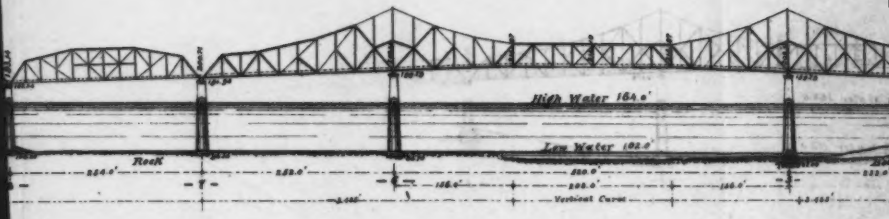




CHIEF ENGINEERS,
Ferris, Kaufman & Co.,
OF PITTSBURGH, PA.

RESIDENT ENGINEER,
A. A. Stuart.

PLATE XXVII.



GENERAL ELEVATION.

GENERAL PLAN.

CENTRAL BRIDGE

ACROSS THE

OHIO RIVER

BETWEEN

Cincinnati, O. & Newport, Ky.

Built by The Central Railway & Bridge Co.

1890-91.

- SCALE - 1 inch = 200 Feet.

& Co.,
, - PA.

Contra
The

PLATE XXVII.
TRANS. AM. SOC. CIV. ENG'RS.
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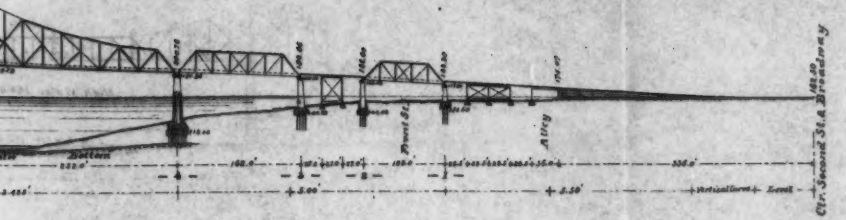
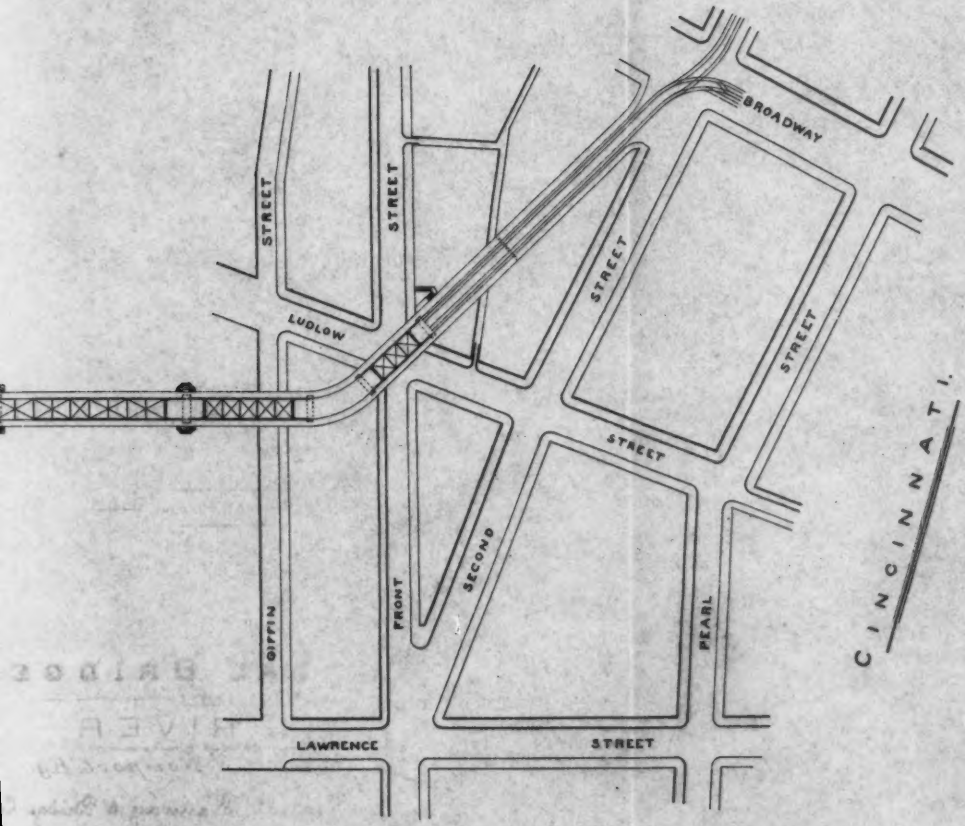
KAUFMAN AND OSBORN ON
CANTILEVER HIGHWAY BRIDGE.





Contractors for the entire structure,
The King Iron Bridge & Manf'g Co.,
OF CLEVELAND, - O.

PLATE XXVIII.
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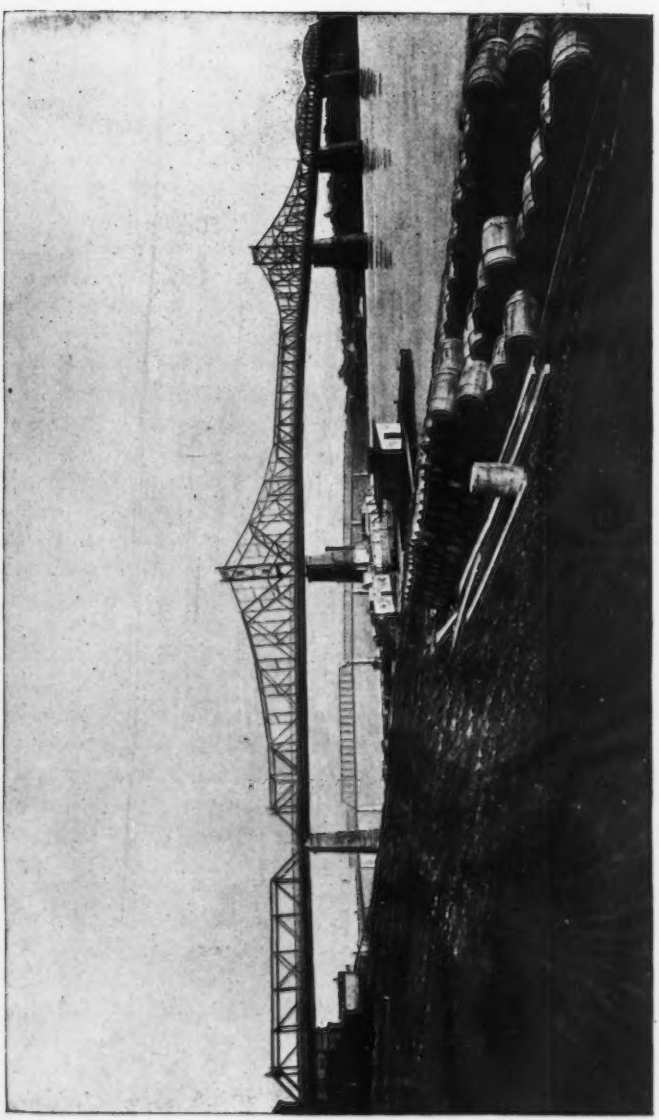
KAUFMAN & OSBORN ON CANTILEVER
HIGHWAY BRIDGE.



eral view is shown in Plate XXVIII. The approaches were located entirely on private property occupied by a large number of buildings of all kinds and descriptions, which were owned and leased by almost an equal number of individuals. All the street and alley crossings are overhead. The maximum grade, which occurs on the Cincinnati end, is 5½ per cent. This grade is necessary to enable the structure to give the clear height over the channel, required by the Act of Congress governing the construction of bridges across the Ohio River. Between the mouth of the Big Sandy River and the Suspension Bridge at Cincinnati, all bridges must have a channel span 500 feet wide in the clear at low water. The lowest part of the channel span must be 100 feet above low water-mark, and 40 feet above local highest water. The highest water known at Cincinnati prior to 1883, the time of the passage of the act, was in 1832, when the river reached the height of 64 feet on the Government gauge. In 1884 the river rose to the unprecedented height of 71 feet and three-quarters of an inch. Since that time the law has been construed by the Government authorities to refer to the flood of 1832 and in fixing the height of channel spans it is only necessary to have 40 feet clear above that flood. Therefore, in the case of the Central Bridge, it was required to have a clear height of 102 feet above low water-mark, which is 2 feet on the gauge.

No active work of construction was done on the bridge until March, 1890, when, through the perseverance of Mr. V. Morris, the Southwestern Agent of the King Bridge Company, arrangements for its construction were perfected with the Company which he represented. This company was to construct the bridge complete in all details by January 1st, 1891, under the specifications to be prepared by Ferris, Kaufman & Co., who were at this time appointed Chief Engineers of the Central Railway and Bridge Company. On account of the limited time, but twenty-four hours were taken in preparing the specifications, copies of which will be found in the appendices.* The general plans adopted by the Government were of course adhered to, and many items in the original specifications of the company were adopted. On the 31st day of March, 1890, the formal contract was executed. By its terms the King Bridge Company was bound, under heavy forfeiture, to complete the structure satisfactorily to the engineers by the time mentioned.

* See pages 194 to 220 inclusive.

The preliminary estimate of quantities was as follows:

SUBSTRUCTURE:

First-class masonry, Piers 4, 5, 6, 7 and 8	10 420	cubic yards.
" " " " 1, 2, 3 and 9	1 353	"
" " " Pedestals	140	"
Second " " Abutments and Ramps	3 200	"
Concrete	2 100	"
Piles in foundations	20 400	linear feet.
Timber "	152 000	ft. B. M.
Iron	15 000	pounds.
Excavation in coffer-dams	1 750	cubic yards.
" foundations on shore	2 750	"
Filling between ramp walls	3 000	"
Granite paving	2 500	sq. yds.

SUPERSTRUCTURE:

Structural iron and steel	2 500	tons.
Lumber for floor	550 000	ft. B. M.
Hand rail	6 000	linear feet.

Toll houses, gas pipe, etc.

The right of way on the Newport side was practically all secured, but nothing had been done in the way of clearing it at the time of the closing of the contract, and no property had been obtained on the Cincinnati side. The undertaking of the King Bridge Company to complete this work in nine months was large; but, at the same time, it seemed that the contract could be successfully carried out if proper energy were used by all, and if no extraordinarily unfavorable circumstances should arise. The company, in April, sub-let the contract for the substructure and paving of approaches to Mr. J. Le Duke, of Berea, Ohio. In accordance with this contract the various parts were to be completed, as follows:

Piers Nos. 1, 2, 3, 4, 6, 7, 8, 9, pedestals and approach masonry not later than October 1st, 1890; Pier No. 5 not later than October 31st, 1890, and all the earth filling, granite paving, flagging and the entire contract not later than December 1st, 1890. The work was to be commenced not later than May 1st, 1890, and the contractor was placed under heavy bonds for the completion of the work as stated. The ordinary clause giving the contractor more time for the completion of his work, on account of delays through causes beyond his control, was

omitted in this contract, and the contractor was given to understand clearly that he was to take all the chances and finish the work in time to enable the erection of the bridge to be completed by January 1st, 1891.

The contract for the iron and steel was given to Messrs. Carnegie, Phipps & Co.; the material was to be delivered to the King Bridge Company at Cleveland within sixty days from the time the order was placed, and this would enable the latter Company to deliver the finished material at the bridge site as agreed upon.

The contract for the erection of the superstructure and the laying of the floor, etc., was awarded to Messrs. Baird Bros., of Pittsburgh, Pa., who were to be given possession of the piers and material at the time shown in Le Duke's contract, and were to complete the work by January 1st, 1891.

For a short time the work in all departments progressed very satisfactorily and according to programme; but soon delays from various causes arose, until finally, as it became evident that the bridge could not be completed as contracted for, the masonry contractors became demoralized; and it required great patience and energy to maintain the prosecution of the work.

The Ohio River has for some miles above and below the bridge site a narrow and tortuous channel; and as the mouth of the Licking River is almost directly opposite the site, it is subject to very rapid and wide fluctuations. The variation between extreme high water and extreme low water is about 69 feet. These conditions conspired to render the foundation work hazardous and expensive. The year 1890 is now noted for the number of floods which occurred in the Ohio River. A hydrograph showing the stage of the water, the condition of the weather and the temperature for each day during the construction of the work, will be found on Plate XXIX. This will indicate one of the great difficulties under which this work was carried on. Usually during the summer and fall months the water stage at Cincinnati is below 15 feet, with an occasional rise above that point, but during 1890 the reverse was the case. There were about five weeks of low water. During this period Piers 6 and 7 were fairly started, but Pier 5 was caught by the high water. The delay in completing this pier prevented the opening of the bridge until August, 1891, some eight months behind time.

The year 1890, as is well known, excelled all other years in the product of steel and iron, and great difficulty was encountered in getting the structural material to the bridge shops. The mills failed in a great measure to complete their contract in this case, and the time for the delivery had elapsed by several months before the King Bridge Company obtained all the material they had ordered. The delay from this cause was not particularly noticeable on account of the fact that the masonry was not finished.

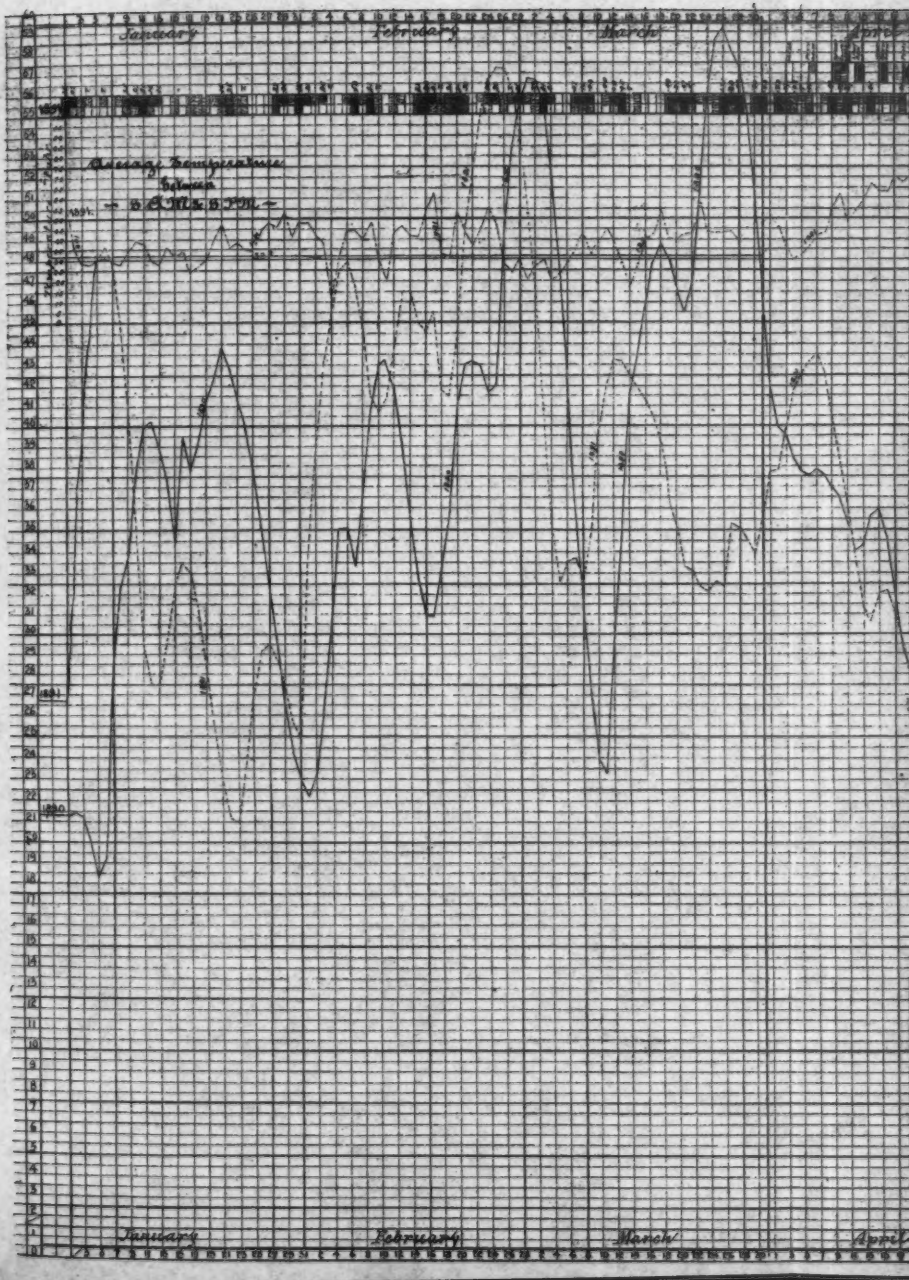
Surveys.—Surveys and locations for the construction work, and, in fact, all the substructure work, were made under the direction of Mr. A. A. Stuart,* M. Am. Soc. C. E., who was Resident Engineer, and Mr. L. V. Rice, Assistant Engineer, to both of whom much credit is due for the accuracy and excellency of the work. The general situation was favorable for accurate triangulation work, and to this is largely attributable the excellent results obtained.

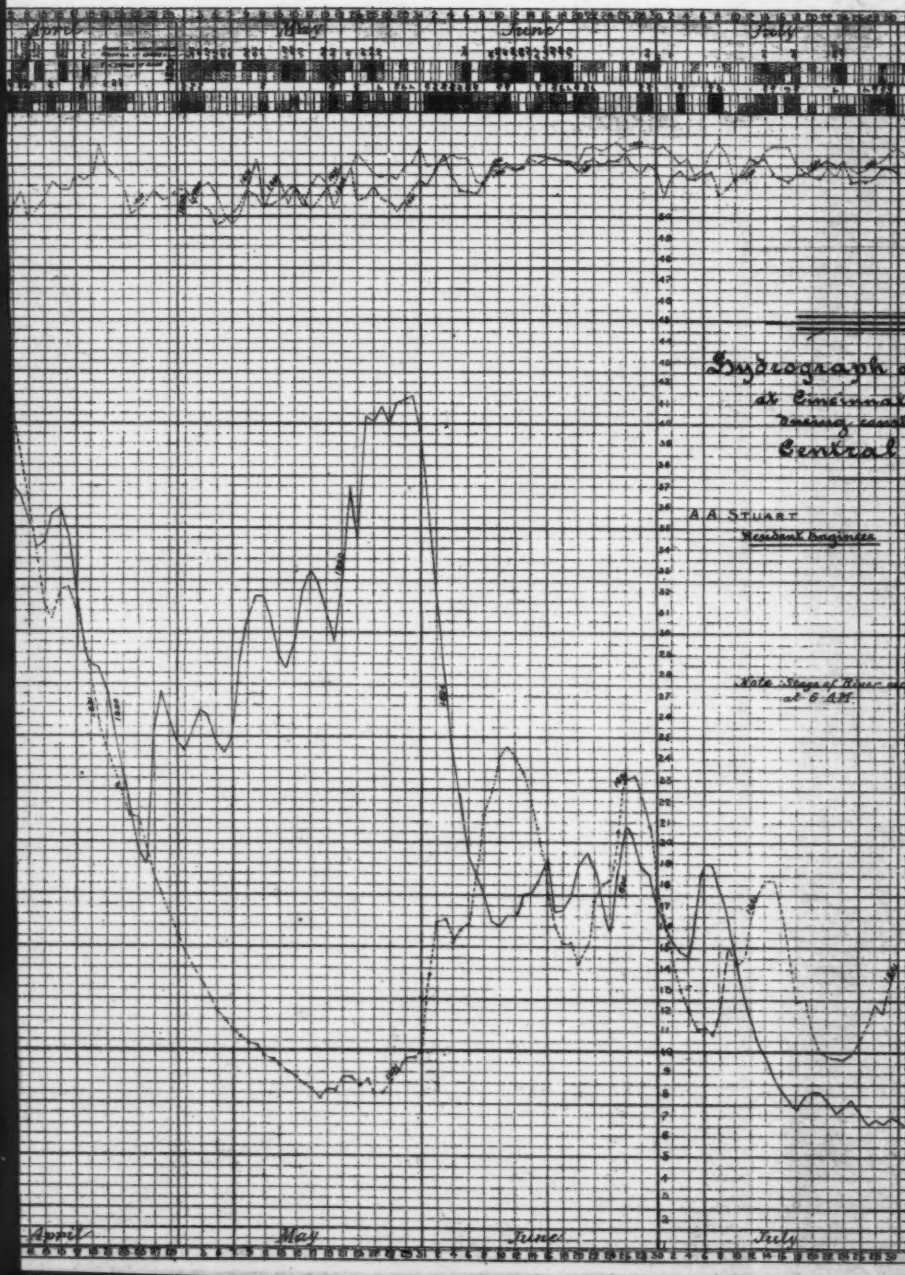
A test of the accuracy of the field operations was made by computing the distance between the base line on the Cincinnati side and a point on the bridge axis common to the two base lines on the Newport side, using the three triangles formed on the three base lines, and the results were respectively as follows: 1 696.076 feet; 1 696.074 feet; 1 696.053 feet. With the assurance of accuracy of field work which these results gave, the remaining elements of the triangles were computed ready for use in locating the river piers.

Masonry.—The length of the structure between the termini is 2 966 feet. Beginning in Cincinnati, this distance is made up as follows:

Granite paving and masonry ramp	285	feet.
Steel viaduct	151	"
One truss span across Ludlow street	108	"
Viaduct	81	"
One truss span	162	"
Cincinnati cantilever arm	252	"
Two river arms and suspended span	520	"
Newport cantilever arm	252	"
Two truss spans, each 254 feet	508	"
Steel viaduct	319	"
Granite paving and masonry ramp	328	"
Total	2 966	"

*The reports of Mr. Stuart were frequently drawn upon in preparing this paper.


January
February
March
April
Average Temperature
January
February
March
April


April
May
June
July
Hydrograph
at Cincinnati
Ohio
Central
A. A. STUART
Resident Engineer
Note: Stage of River
at 6 A.M.
April
May
June
July

August September October November

graph of the Ohio River
Cincinnati, O. 1890-95
showing condemnation of
Central Bridge

FERRIS, KAUFMAN & CO.
Chief Engineers
PITTSBURGH PA.

of River corresponds to Water Works Gauge
A.M.

August September October November

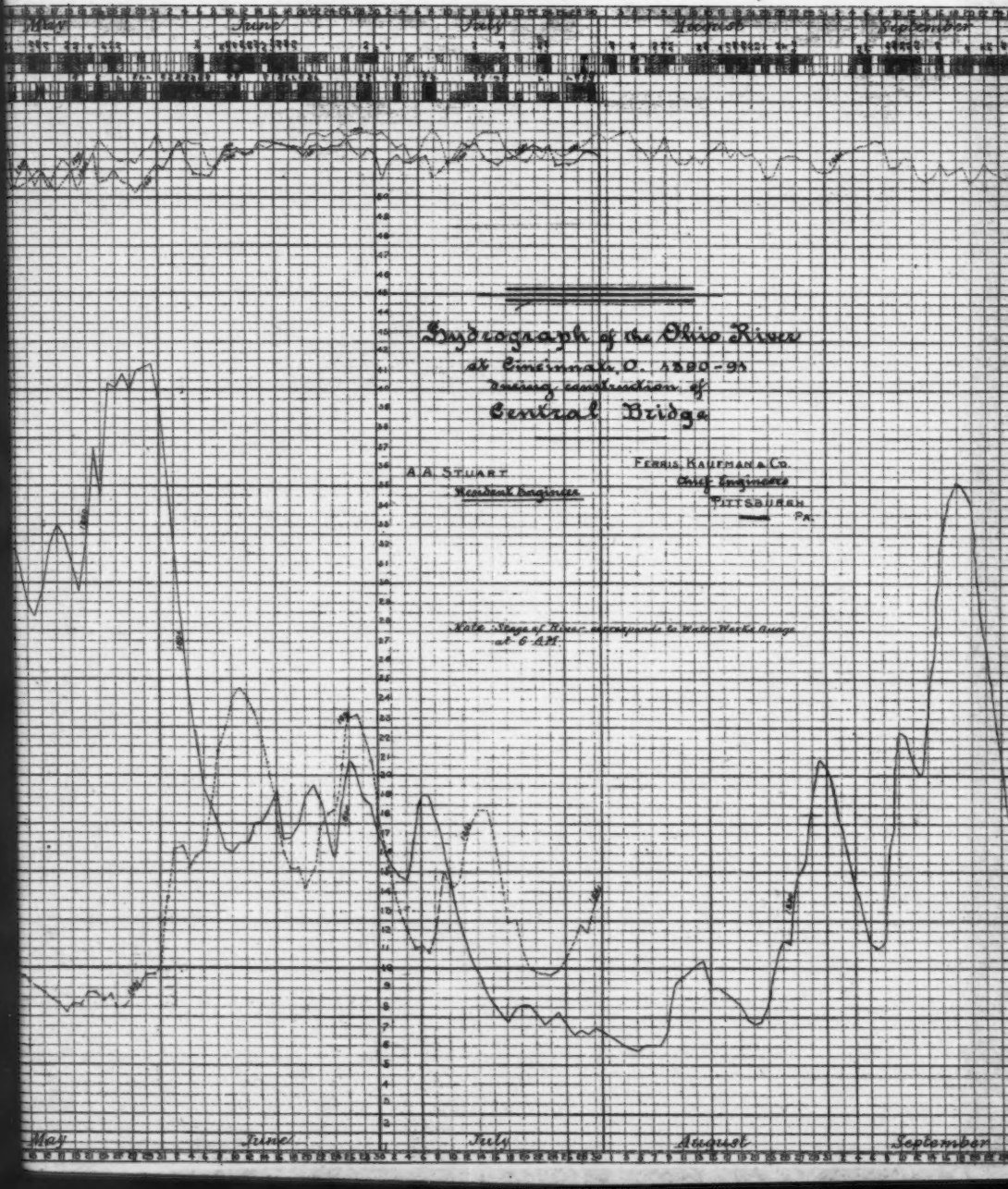

May
June
July
August
September
Hydrograph of the Ohio River
at Cincinnati, O. 1890-95
showing construction of
Central Bridge
A.A. STUART
Resident Engineer
FERRIS, KAUFMAN & CO.
Chief Engineers
PITTSBURGH
PA.
Note: Stages of River correspond to Water Works Gauge
at 6 A.M.

PLATE XXIX.
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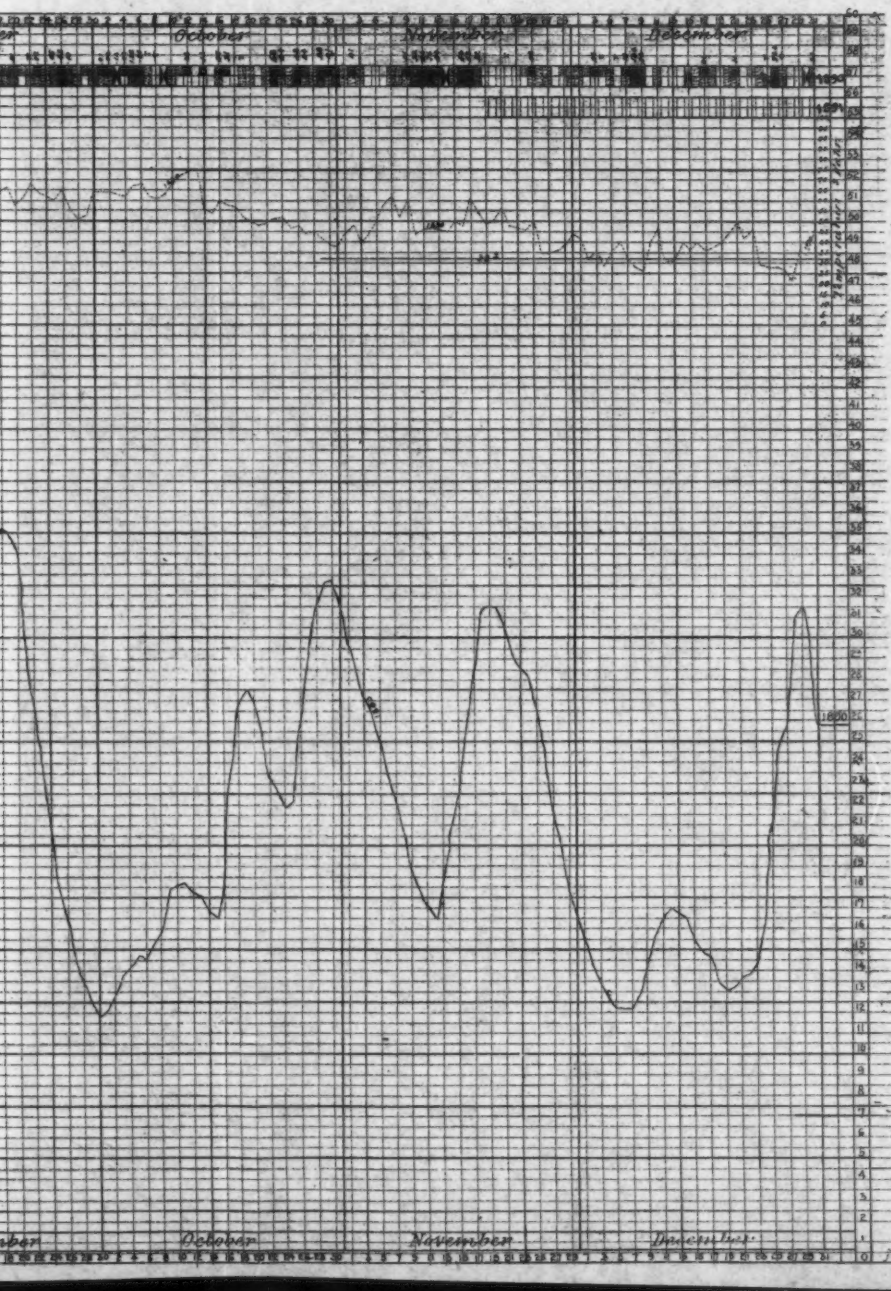
KAUFMAN AND OSBORN ON
CANTILEVER HIGHWAY BRIDGE.



The superstructure is supported by two abutments, twenty-eight pedestals and nine piers. The abutments and ramp walls are built of second-class masonry and entirely of Ohio River freestone, except the coping, which is of Berea sandstone. All the pedestal piers are built of first-class masonry. All the cement used on the work, with the exception of a few barrels of Portland for main coping stone and pointing, was Louisville cement, and was tested at the mills by Messrs. Mead & Shaw, Cement Inspectors, of Louisville, and all accepted barrels which were shipped were branded by them. The ramp walls, abutments and pedestals being far removed from the water, are founded on the natural earth upon a course of concrete 2 feet thick, the material affording ample resistance to bear the superimposed loads without the aid of piling.

Piers Nos. 1, 2, 3 and 9 are similar in all respects, except as to size and height, and are all founded on piles driven to a firm resistance from short blows of a hammer weighing 4 000 pounds. The foundation beds were from 7 to 10 feet deep, and after sawing the piles off 18 inches above the bottom of the pits, concrete was put in, varying in thickness from 3 to 4½ feet, thoroughly imbedding the piles in a plastic mass upon which the foundations and footing courses were started. These piers were built entirely of Ohio River freestone, except the coping which was Bedford oolitic limestone. They are rectangular in plan throughout their height, battering one-half inch to the foot, and as they stand above the average high water, no difficulties were encountered in constructing their foundations.

Piers Nos. 4 and 8 are similar in construction, but different in kind of foundations and dimensions. Pier No. 4 rests upon one hundred and fifty piles, driven to solid rock, having heavy cast-iron shoes, the points of which were seated in the rock by repeated light blows from the hammer. They were cut off 18 inches above the bottom of the foundation bed, and their heads were imbedded in concrete 3 feet 6 inches thick. Upon this the foundation footing courses, four in number, were laid.

Pier No. 8 is located at about extreme low water-line on the Kentucky shore, and rests upon solid rock. This foundation was begun July 14th, 1890, and the laying of masonry was begun July 27th, the stage of the river being about 7 feet. A clay dam was built around the site of the pier, and with this protection from the water the exca-

vation was made. A pit 18 inches deep was excavated in the rock, and in this the footing courses were started.

These two piers have semicircular nosings up to the belt course, where they are contracted in length and become rectangular in plan. From the foundation to the belting course the face work is built of Berea sandstone, with concrete backing. The face stones were laid in Flemish bond, headers and stretchers alternating with each other in every course. The concrete backing was put in as fast as the face stones of each course were laid, and was allowed twelve hours to set before any masonry was laid upon it. Above the belting course these piers are built entirely of Ohio River freestone, except the coping, which is of Bedford limestone. The table on next page gives the complete record of the building of Pier No. 4, and it is believed by the writer that this is the first instance that such a record has been kept.

Each face stone and each backing stone was measured and its contents calculated. The sum of the two was deducted from the contents of the full course and the balance was taken as the amount of mortar in that course. In this way the column giving the cubic yards of mortar in all joints was obtained. The number of barrels of cement used for the face work and backing was obtained by actual count. The column giving the number of barrels of cement per cubic yard of backing, gives also the amount of cement in a yard of concrete, as the backing is concrete. The cost of laying up to the starling course refers only to the face stones.

Piers Nos. 5, 6 and 7 are similar in construction and are located in the river. Piers 6 and 7 are founded on the solid rock, and their foundations were put in without difficulty by the use of single wall coffer-dams. The solid rock bed in the river at this point has no deposit upon it, and in landing the coffer-dams it was necessary first to sink a crib composed of timbers and stone above the pier site in order to hold the coffer-dam in place. The coffer-dams for Piers 5, 6 and 7 were all alike in construction, being rectangular in plan and 30 x 70 feet in size out to out. The walls were built of horizontal courses of 12 x 12-inch timber bolted together for a height of 6 feet, and above this the walls were composed of 6 x 4-inch stuff. At intervals of 8 feet along the longitudinal walls 12 x 12-inch vertical timbers were bolted, and into each pair of verticals 12 x 12-inch horizontal struts were dovetailed.

CENTRAL BRIDGE, CINCINNATI–NEWPORT.—BUILT 1890–91.—DATA FROM PIER NO. 4.

Number of course.	Mean size of course out to out.	Thickness of stone.	Cubic yards of face masonry.	Cubic yards of backing.	Cubic yards of mortar in all joints.	Face work. Percentage of whole.	Barrels cement per yard of face work.	Barrels cement per yard of backing.	Cost per yard for laying, inc. cem. and sand.	Remarks.
	Feet.					Per cent				
1	23.18 x 59.18	1′ — 7″	30.90	46.05	1.45	40	0.26	1.70	$1.90	Footings, Polygonal ends—concrete backing.
2	21.35 x 57.35	1′ — 7″	29.35	39.67	1.05	43	0.37	1.64	1.90	
3	19.44 x 55.18	2′ — 8″	53.34	47.37	1.30	53	0.32	1.52	1.90	
4	16.44 x 52.18	2′ — 4″	32.35	38.95	0.93	45	0.37	1.79	1.90	
5	13.69 x 49.44	2′ — 2″	28.40	22.70	0.88	58	0.35	1.68	1.65	
6	13.50 x 49.25	"	26.70	23.89	0·79	53	0.30	1.65	"	
7	13.31 x 49.06	"	28.86	20.79	0.87	58	0.21	1.80	"	
8	13.12 x 48.87	"	28.31	20.48	0.87	58	0.25	2.00	"	Concrete backing.
9	12.93 x 48.68	"	28.74	19.25	0.87	60	0.24	1.71	"	
10	12.74 x 48.49	"	27.85	19.34	0.86	59	0.25	1.94	"	
11	12.55 x 48 30	"	26.48	19.90	0.83	57	0.30	1.71	"	
12	12.36 x 48.11	2′ — 1″	26.42	17.66	0.84	60	0.26	1.81	"	
13	12.18 x 47.93	2′ — 0″	23.99	17.43	0.80	58	0.27	1.71	"	
14	12.00 x 47.75	"	25.11	15.69	0.82	62	0.24	1.60	"	Semicircular ends.
15	11.81 x 47.56	"	24.78	15.29	0.81	62	0.24	1.77	"	
16	11.62 x 47.37	2′ — 1″	25.74	15.17	0.83	63	0.31	1.71	"	
17	11.43 x 47.18	"	24.84	15.25	0.82	62	0.36	1.67	"	
18	11.24 x 46.98	"	25 50	14.05	0.75	64	0.33	1.74	"	
19	11.05 x 46.80	"	24.71	14.02	0.80	64	0.42	1.64	"	
20	10.86 x 46.61	"	24.61	13.47	0.80	65	0.45	1.74	"	
21	10.66 x 46.42	2′ — 0″	24.07	11.01	0.80	69	0.29	1.63	1.65	
22	11.76 x 47.50	1′ — 10″	23.11	12.71	6.20	65	0.42		1.96	Starling.
23	10.70 x 46.44	1′ — 5″	15.09	10.49	4.99	59	0.47		"	Hood courses.
24	10.58 x 35.66	"	15.17	8.46	4.56	64	0.53		"	
25	10.46 x 35.52	1′ — 6″	10.02	12.13	8.40	45	0.53		"	
26	10.34 x 35.37	1′ — 6″	12.25	7.50	3.50	62	0.53		"	Rectangular in plan.
27	10.22 x 35.23	1′ — 5″	11.81	7.54	3.64	61	0.55		"	
28	10.10 x 35.08	1′ — 4″	11.39	6.60	3.23	63	0.56		"	
29	10.00 x 34.96	1′ — 4″	11.68	6.08	2.83	66	0.68		"	
30	9.87 x 34.84	1′ — 5″	12.53	5.93	2.03	67	0.64		"	Courses 22 to 38 inclusive have stone backing.
31	9.75 x 34.72	1′ — 3′	10.16	5.92	2.52	63	0.55		"	
32	9.62 x 34 61	1′ — 4″	10.23	6.62	3.77	61	0.56		"	
33	9.48 x 34.49	1′ — 5″	11.07	6.62	2.98	63	0.56		"	
34	9.35 x 34.37	1′ — 6″	11.36	7.00	4.30	62	0.54		"	
35	9.25 x 34.25	1′ — 3″	9.21	5.92	2.77	61	0.72		"	
36	9.15 x 34.15	1′ — 3″	9.18	5.75	3.00	62	0.60		"	
37	9.03 x 34.03	1′ — 3″	8.88	5 84	2.95	60	0.47		1.96	
38	10.00 x 35.00	1′ — 6″	13.80	6.16	2.99	69	0.50		2.06	Sub-coping.
39	11.00 x 36.00	1′ — 6″	21.82	.....	0.72	100	0.50		2.06	Coping.

Included with face work.

Amount of cement per yard of face work, courses 1 to 21 inclusive = 0.30 barrels.
" " " " " " concrete " " " " " = 1.71 "
" " " " " " masonry " 22 " 39 " = 0.56 "
Percentage of face work in " 1 " 21 " = 55 per cent.
" " " " " " " 22 " 39 " = 62 "

NOTE.—From the foundation to the starling course, the masonry is built of Berea sandstone with concrete backing, but above this it is built entirely of Ohio River freestone. All face stones were required to have a width of 1½ times their thickness, and below the starling they were laid with Flemish bond. All mortar was mixed 1 cement to 2 of sand, and the concrete was mixed 1, 2 and 4, the latter being broken stone. Louisville cement was used throughout.

The bottom edges of the walls were padded with cotton waste 6 inches thick, held in place by cotton ducking. The coffer-dams were towed into place without bottoms and sunk by loading them with stone intended for use in the piers. On July 10th, the crib for Pier No. 7 was located when the water was about 14 feet high and the current very swift. On the 11th of July the coffer-dam was towed into place and sunk, the stage of water being about 10 feet. After sinking it was discovered to be uneven, the southeast corner being about 1 foot higher than the others, a 12 x 12-inch stick of timber having lodged in this corner. This was removed by a diver. Considerable difficulty was experienced in pumping out this coffer-dam, the cotton ducking having been torn out in a number of places in launching, causing leaks which were finally stopped by throwing in bags of sand. This work consumed considerable time, but by July 22d a bed 2.7 feet deep was excavated in the rock and the first course of masonry begun.

On July 20th the coffer-dam of Pier No. 6 was located after considerable trouble on account of a very swift current. The stage of the water was at this time 8 feet, and a week was consumed in stopping the leaks around the bottom of this coffer-dam. After excavating a bed 3.8 feet deep in the rock, masonry was started August 1st.

On August 10th the coffer-dam for Pier No. 5 was located and sunk in position, but as the rock bottom at this point was overlaid with about 2 feet of river silt, much time was taken in an effort to make the bottom edge of the coffer-dam tight. Before this was accomplished the river began to rise and all operations were suspended on August 26th. From this time the water stage fluctuated between 11 and 20 feet, until on September 18th it reached a 35-foot stage, which was never known to occur before in the month of September. The condition of the work at this time, starting from the Cincinnati end, was as follows: The right of way from Broadway to Giffin Street had not been entirely obtained, many suits of condemnation having made the work of securing it extremely tedious, but it had been so far obtained as to allow the foundation of Pier No. 1 to be put in; also Piers Nos. 2 and 3. On Pier No. 4 nothing had yet been accomplished, owing to litigation as to the right of the company to condemn property at this point. Pier No. 5, as stated above, had not yet been started, but a coffer-dam had been sunk. Pier No. 6 had 33 feet of masonry yet to be laid to complete it. Pier No. 7 had yet 30 feet of masonry to be laid

on it. To complete Pier No. 8, 36 feet of masonry were required. Pier No. 9 was completed. The pedestals on the Newport side, with the exception of a few cap stones; the abutment on the Newport side, and most of the ramp walls were completed. No filling between the same, however, had yet been done. Out of a total of 13 000 cubic yards of masonry, 7 000 yards had been laid.

During the whole season to this date, there had been but five weeks in which the stage of the water was below 10 feet, and only for one day was the water below a 6-foot stage, and during this time there were a number of very rainy days. The masonry contractors were becoming very much demoralized, and they realized that it was impossible for them to complete their work by the specified time. Considerable iron had been delivered on the ground and the contractors for the erection were on hand ready to proceed with their work.

As winter was rapidly approaching it became obvious that if the work was to be finished approximately on time, it was absolutely necessary that the river work be completed first. The condition of Pier No. 5 and the condition of the river made it clear that some radical move had to be made. It was therefore determined on September 15th to use the pneumatic process in founding Pier No. 5, notwithstanding the fact that bed-rock was only about 7 feet below low water. Plans and specifications for a caisson were made as rapidly as possible.

The plan of the caisson is shown on Plate XXX and the specifications in Appendix 3. It was 12 feet high from the shoe to top of the deck, with a coffer-dam about 24 feet high, so that the work could be prosecuted in a 24 to 26-foot stage of water after the caisson was landed on the rock bottom of the river. An examination of the records gave sufficient reasons to expect that the work could be thus carried on without interruption during the early winter months and that the pier could be finished before very cold weather set in.

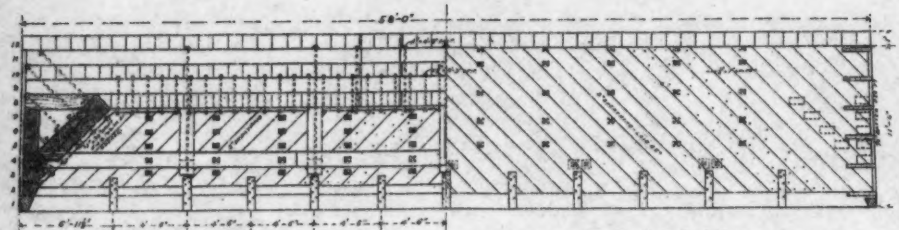
Work was accordingly begun on the caisson September 23d, and on October 17th it was launched. A pressure plant belonging to Messrs. Sooysmith & Co. was obtained from Louisville, and this firm was contracted with to do the work. At this time another freshet came in the river, and as this pier stands in the main channel where the current is swiftest, it was deemed unwise to attempt to locate it until the water would fall somewhat. On November 7th, with about 22 feet of water in the river, the caisson was towed into position, and on November 9th

the work of laying masonry began. Owing to the rapid fluctuations in the water level, the cutting edge was not landed on the bottom until November 28th, and on November 29th the air compressor was started and the work of excavating and removing material in the working chamber was begun at once.

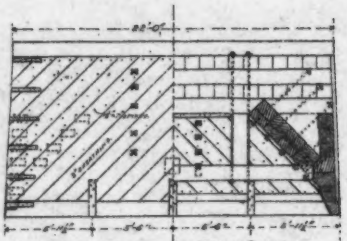
During the month following this date, the river stage permitted the work of excavating and sinking of caisson to go on without interruption. On account of the material being rock, the laying of masonry was subject to careful regulation so that the cutting edge would not be liable to injury from excessive pressure. On December 27th, 1890, the caisson had penetrated into the rock about 5 feet and had 38 inches yet to go. Seven full courses of masonry had been laid, and about ten or twelve working days were yet necessary to complete the sinking, seal the air chamber, and to bring the masonry up high enough to be out of the way of a 32-foot stage of water. On that day, however, the river rose above the shafts and suspended all operations.

At this time the condition of the work was such that the contractors could work in a 28-foot stage of water by the aid of the coffer-dam, but from December 27th until April 18th, 1891, as shown on the hydrograph, at no time did the stage of the river allow the contractors to do anything toward the completion of Pier No. 5. On April 28th the sinking was completed, and by May 8th the working chamber and shafts were filled with concrete. Laying masonry was not resumed until April 28th, from which time it was continued without further interruption from high water.

The caisson in this foundation was of the Morrison type, except that the iron shoe was omitted. The sinking was accomplished without accident or injury to any of the men engaged on it, and required seven hundred and twenty hours actual working time to penetrate 8 feet into the solid rock, or an average of 3.2 inches for each twenty-four hours. The rock penetrated consisted of ledges of fairly hard shaly formation alternating with thin ledges of hard fossiliferous limestone. Where first struck it was not well adapted to make a good foundation, and in order to get the deck of the caisson 3 feet below extreme low water, it was necessary to penetrate the rock 8 feet. The last piece of coping was set on Pier No. 5 at 9 A.M., June 18th, 1891, entirely completing the substructure within twelve months from the time of beginning the work.



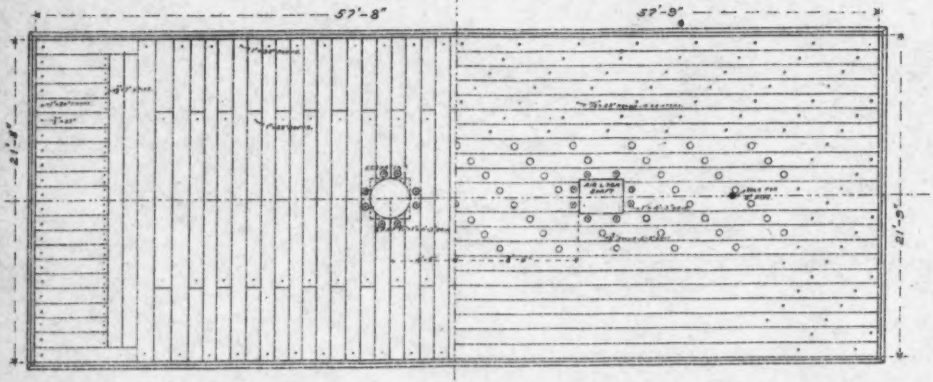
LONGITUDINAL SECTION.



SIDE ELEVATION.

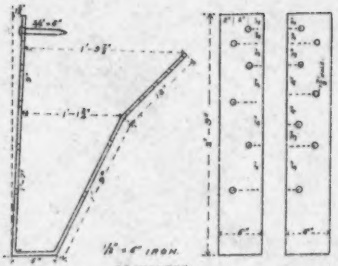
END ELEVATION.

CROSS SECTION.



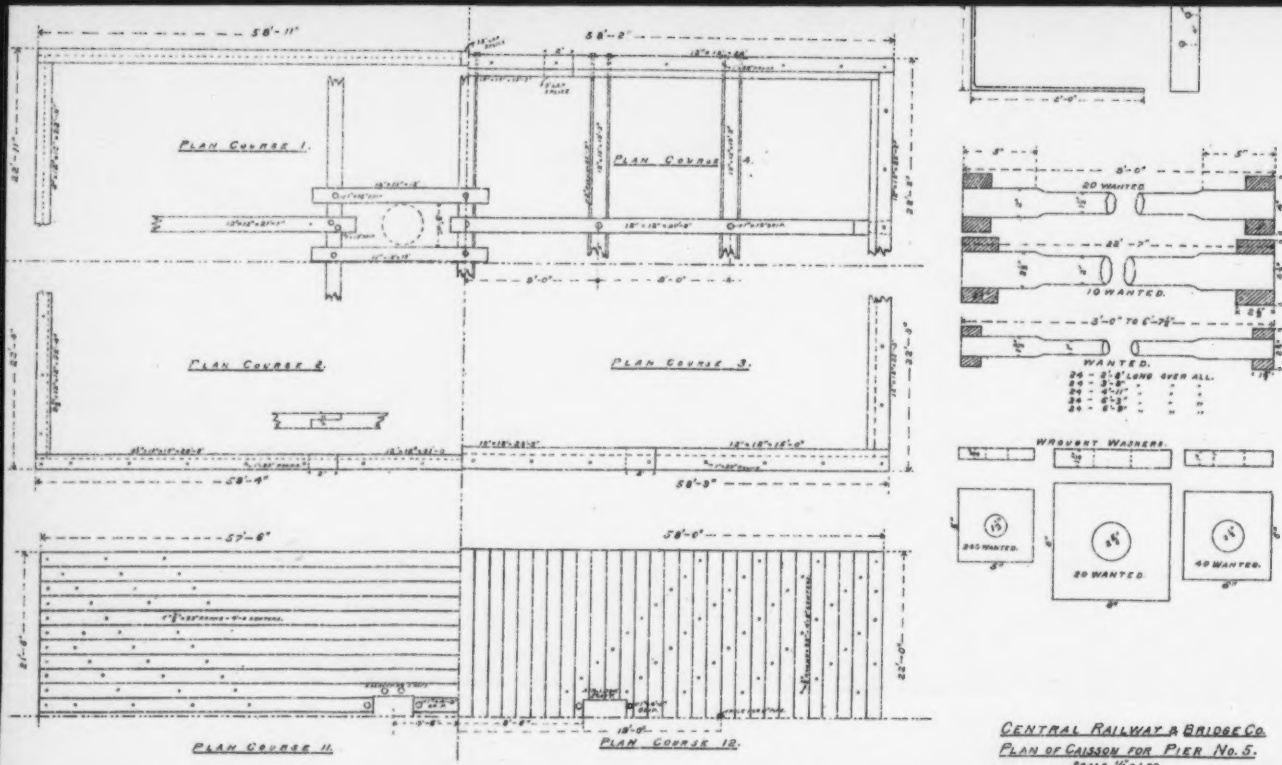
PLAN COURSE B.

PLAN COURSE 3.



1/8" × 6" IRON.
28 WANTED
SHOE STRAP.

24 WANTED.
CORNER STRAP.
1/8" × 4" IRON.



CENTRAL RAILWAY & BRIDGE Co.
PLAN OF CAISSON FOR PIER No. 5.
SCALE, 1/4" = 1 FT.
SEPT. 20TH 1924.

PLATE XXVIII
TRANSAMSOC.CIV.ENG.
VOL.XXVIII
KAUFMAN AND OSBORN
CANTILEVER HIGHWAY

LATE XXX.
SOC. CIV. ENGRS.
XXVII. Nº 545.
AND OSBORN ON
HIGHWAY BRIDGE.

Piers Nos. 5, 6 and 7 have semicircular nosings up to the belting course. The under side of each is set at water stage 66 feet, and they, from the belting course to the under side of the coping, are built entirely of Ohio River freestone. The coping is 2 feet thick on Piers Nos. 5 and 6, and 18 inches thick on the other piers, all being of Bedford oolitic limestone, from Bedford, Ind.

The following table gives data of the construction of Pier No. 5. It gives the size and thickness of each course, number of cubic yards in the face stones and number of cubic yards of backing; also number of cubic yards of mortar in bed joints, the amount of cement required, and it also shows the cost of laying masonry, including sand and cement.

CENTRAL BRIDGE, CINCINNATI–NEWPORT.—BUILT 1890–91.—DATA FROM PIER NO. 5.

Number of course.	Mean size of courses out to out.	Thickness of stone.	Cubic yards of face masonry.	Cubic yards of backing.	Cubic yards of mortar in bed joints.	Face work, percentage of whole.	Barrels cement per yard of masonry.	Cost per yard for laying, including sand and cement.	REMARKS.
	Feet.	Feet.				Pr. ct.			
1	19.50 x 55.50	2.25	33.29	52.61	1.620	30.8			Foot'gs. Polygonal ends.
2	19.00 x 55.00	2.16	32.24	48.09	1.550	40.0			" "
3	19.00 x 55.00	"	28.54	51.01	1.525	36.0			Semicircular "
4	17.25 x 53.57	"	30.03	38.79	1.325	43.6			
5	17.07 x 53.39	2.08	28.94	36.46	1.310	44.0			
6	16.90 x 53 22	"	29.26	35.30	1.280	45.0			
7	16.73 x 53.05	"	27.33	36.39	1.260	43.0			
8	16.56 x 52.88	"	29.47	33.41	1.250	47.0			
9	16.39 x 52.71	2.00	28.50	31.23	1.245	47.7			
10	16.22 x 52.54	"	26.00	32.94	1.220	44.1			
11	16.04 x 52.36	2.16	29.40	33.38	1.207	47.1			
12	15.86 x 52.18	"	29.77	33.01	1.194	48.0			
13	15.68 x 52.00	2.08	27.30	31.34	1.180	46.5			
14	15.50 x 51.82	"	25.25	32.67	1.165	43.6			
15	15.32 x 51.64	"	27.48	30.43	1.150	47.4	0.3289		
16	15.15 x 51.47	2.00	26.72	27.39	1.130	49.4		$1.127	
17	14.98 x 51.30	"	28.16	25.20	1.110	52.7			
18	14.81 x 51.13	"	25.56	27.05	1.095	48.4			Semicircular ends.
19	14.64 x 50.96	"	25.27	26.59	1.081	48.7			
20	14 47 x 50.79	"	25.00	26.12	1.066	48.8			
21	14.30 x 50.62	"	25.42	24.95	1.050	50.4			
22	14.13 x 50.45	"	25.22	24.40	1.035	50.8			
23	13.96 x 50.28	1.83	21.86	22.88	1.020	49.7			
24	13.79 x 50.11	"	23.38	20.38	1.005	52.8			
25	13.62 x 49.94	"	21.90	21.50	0.990	50.4			
26	13.45 x 49.77	"	20.78	21.96	0.973	48.0			
27	13.28 x 49.60	1.75	21.90	18.34	0.960	54.4			
28	13.14 x 49.46	"	22.52	17.19	0.940	56.6			
29	13.00 x 49.32	1.83	22.35	18.65	0.933	52.0			
30	12.85 x 49.17	"	20.63	20.07	0.920	50.6			
31	12.69 x 49.01	1.66	20.25	16.00	0.907	62.4			
32	12.54 x 48.86	1.58	18.47	15.53	0.897	54.0			
33	12.40 x 48.72	1.83	22.25	16.38	0.882	58.7			

CENTRAL BRIDGE, CINCINNATI–NEWPORT.—BUILT 1890–91.—DATA FROM PIER No. 5—(Continued).

Number of course.	Mean size of courses out to out.	Thickness of stone.	Cubic yards of face masonry.	Cubic yards of backing.	Cubic yards of mortar in bed joints.	Face work, percentage of whole.	Barrels cement per yard of masonry.	Cost per yard for laying, including sand and cement.	REMARKS.
	Feet.	Feet.				Pr. ct			
34	13.32 x 49.64	1 83	23.33	17.54	0.860	57.0			Starling course,
35	12.26 x 48.20	1.42	17.77	11.61	0.863	61.0		$1.121	
36	12.14 x 46.20	"	16.77	10.79	0.798	60.0			Hood courses.
37	12.02 x 39.89	"	15.05	8.10	0.677	65.0			
38	11.90 x 35.80	"	13.75	8.72	0.660	61.0			
39	11.78 x 35.78	1.33	14.33	6.48	0.651	69.0			
40	11.66 x 35.66	"	12.27	8.21	0.641	60.0			
41	11.54 x 35.54	1.42	14.81	6.79	0.632	68.5			
42	11.42 x 35.42	1.33	13.85	6.12	0.625	69.0			
43	11.30 x 35.30	"	12.55	7.04	0.616	64.0			
44	11.19 x 35.19	"	12.32	7.13	0.608	63.0	0.3010		
45	11.07 x 35.07	"	12.73	6.44	0.598	66.0		$1.470	
46	10.96 x 34.96	"	12.21	6.71	0.592	65.0			Rectangular in plan.
47	10.84 x 34.84	"	13.19	5.46	0.583	71.0			
48	10.73 x 34.73	"	13.70	4.71	0.575	74.0			
49	10.61 x 34.61	"	11.96	6.17	0.567	66.0			
50	10.50 x 34.50	"	12.05	5.95	0.560	67.0			
51	10.38 x 34.38	"	12.36	5.29	0.550	70.0			
52	10.27 x 34.27	"	12.33	5.01	0.540	71.0			
53	10.16 x 34.16	"	12.85	4.28	0.537	75.0			
54	10.05 x 34.05	1.16	10.37	4.33	0.530	70.0			
55	11.00 x 35.00	2.00	20.63	7.89	0.525	72.0	0.348	$1.500	Sub-coping.
56	12.00 x 36.00	"	32.00	None.	0.594	100.0	0.349	1.536	Coping.

Percentage of face work in courses.....1 to 33 inclusive = 46.42 per cent.
" " " " .....34 to 54 " = 65.81 "

NOTES.—From top of caisson to the starling course the masonry is built of Berea sandstone with Ohio River freestone backing; but above this it is built entirely of Ohio River freestone.

All face stones were required to have a width of one and one-quarter (1¼) times their thickness.

All mortar was mixed 1 cement to 2 of sand, Louisville cement being used throughout.

Cost of cement and sand rated at $1.20 per barrel of cement used.

LABOR PER DAY OF TEN HOURS RATED AT.

Foreman	$4 00
Mason	3 25
Cutter	3 50
Engineer	2 00
Common labor	1 50

It is very interesting to note in connection with this table the ratio of increase of the cost of laying as the pier increases in height. On comparing the table for Pier No. 4 with that for Pier No. 5, it will be noted that the cost of laying is higher in the former than in the latter. The reason for this is that Pier No. 4 was constructed during the winter months and the foreman in charge was not so expert as the one

on Pier No. 5. The writer believes that the information in the table is valuable, and if the cost of quarrying and cutting for the various building stones were known, exact estimates of the cost of bridge masonry could readily be made.

Before closing the description of the substructure it may be well to explain why Piers Nos. 4 and 8 were constructed with Berea sandstone face, and concrete heartings, while Piers Nos. 5, 6 and 7 were constructed with the same face stone and Ohio River freestone backing. The original specifications required that the face of the piers should be constructed of limestone obtainable in a quarry near Cincinnati and the backing should be of freestone.

Considerable difference of opinion as to the reliability of this limestone was found among engineers in Cincinnati, and as we had no experience with it, nor time to investigate the matter thoroughly, we decided to take the safe course and not permit its use. A quarry at North Vernon, Indiana, which furnished stone of undoubted quality, could not deliver the quantity as rapidly as necessary. There were then available only the Indiana oolitic limestone and the Berea sandstone quarries, each of undoubted character and of sufficient magnitude to furnish rapidly the quantity of stone required. It was found, however, that the oolitic quarries could give no guarantee of prompt delivery on account of other contracts, and it was decided to obtain the face stone for the important piers from the Berea sandstone quarries. In order to reduce the cost of masonry to that originally specified, the King Bridge Company requested permission to use concrete heartings in the piers in question. The writer had no experience in constructing masonry in this way, but knowing it had been done satisfactorily in several instances of moderate-sized piers, readily granted permission to the contractors to construct Piers Nos. 4 and 8 as requested.

In the absence of information regarding the comparative elasticity of concrete and Berea sandstone, consent to the use of concrete in the interiors of the large Piers Nos. 5, 6 and 7, in which the pressure on the lower course is very great, was withheld. The Berea stone is comparatively soft, and if the concrete should compress more than it did, the periphery of the pier would be subjected to crushing. We did not care to take any chance in the matter. We are not aware of any piers of this size thus constructed except some granite piers on the

Mississippi River, which no doubt could withstand a pressure of this kind.

The contract price for the substructure was as follows:

First.—Filling in abutments on Newport and Cincinnati sides, earth or gravel, per cubic yard, $0.22.

Second.—Excavation of Piers 1, 2, 3, 9, pedestals and abutment foundations, per cubic yard, $0.44.

Third.—Concrete foundations, per cubic yard, $4.40.

Fourth.—Pile foundations, per linear foot, 30.8 cents.

Fifth.—First-class masonry, including the cost of foundation excavation, etc., for Piers Nos. 4 and 8, $11.43 per cubic yard.

Sixth.—First-class masonry, including the cost of foundation complete for Piers Nos. 5, 6 and 7, $12.50 per cubic yard.

Seventh.—First-class masonry, Piers Nos. 1, 2, 3, 9, and pedestals, per cubic yard, $9.90.

Eighth.—Second-class masonry in abutments and ramps, per cubic yard, $7.98.

The quantities of masonry in each part of the work is shown in the following table:

CENTRAL BRIDGE, CINCINNATI-NEWPORT.—BUILT 1890-91.

Pier Number.	Size under Sub-Coping.	Height Over All.	Size at Base of Shaft.	Thickness of Coping.	Cubic Yards of Masonry.	Remarks.
1.....	4'.98 × 29'.98	26'.22	6'.42 × 31'.42	18"	146.18	Square Shaft.
2.....	4'.98 × 29'.98	39'.41	7'.63 × 32'.63	18"	271.68	" "
3.....	6'.00 × 30'.00	47'.00	9'.07 × 33'.07	18"	393.92	" "
4.....	9'.00 × 34'.00	73'.96	13'.77 × 49'.52	18"	1 432.85	Circular Shaft.
5.....	10'.00 × 34'.00	112'.79	17'.34 × 53'.66	24"	2 357.61	Circular Shaft, inc. Caisson in H't.
6.....	10'.00 × 34'.00	104'.09	17'.80 × 54'.16	24"	2 475.59	Circular Shaft.
7.....	9'.00 × 34'.00	93'.43	16'.00 × 51'.75	18"	1 974.12	" "
8.....	7'.00 × 32' 00	87'.14	13'.47 × 46'.83	18"	1 393.30	" "
9.....	7'.00 × 32' 00	37'.28	9'.58 × 34'.58	18"	330.06	Square Shaft.
Cin'ti Ramp.	.....	.....	.....	12"	663 03	.....
Newport Ramp.	.....	.....	.....	12"	1 510.23	.....

Superstructure.—The superstructure of this bridge consisted of the following spans as shown on the general lay-out:

Viaduct spans: one span, 36 feet; four spans, 28 feet 6 inches; one through span, 108 feet center to center of piers; three spans, 27 feet each; one through span, 162 feet center to center of piers; one cantilever arm, 252 feet; one cantilever arm, 156 feet; one suspended span, 208 feet; one cantilever arm, 156 feet; one cantilever arm, 252 feet; two through spans, 254 feet each. Viaduct spans: one, 55 feet; three, 30 feet; three, 61 feet; one, 29 feet; and one, 50 feet. The clearance is 16 feet above the top of roadway, and the clear width between trusses is 24 feet.

The specifications governing the construction of this portion of the work are very full, and contain a number of features which will be discussed by F. C. Osborn, M. Am. Soc. C. E., in his paper on the cantilever span of the bridge. A copy of the specifications will be found in Appendix 2. Full information in regard to loads and unit stresses allowed, the character of the material used and the required tests will be found therein.

The inspection of the material at the mills, the work at the bridge shops and the erection was conducted by the firm of G. W. G. Ferris & Co., of Pittsburgh. About six hundred and fifty tests, taken from rolled sections, were made. The material proved to be of first-class character, all that was accepted falling within the limitations of the specifications. The full-sized eye-bars tested all broke in the body of the bar with the exception of one which failed in the head on account of a flaw. The defective head was cut off and another put on. Upon a re-test this bar broke in the body of the bar.

CENTRAL BRIDGE.—DETAILED WEIGHTS, ETC.

Superstructure:

	Pounds.
36-foot span and bent 1	26 640
28-foot 5-inch span	11 545
Two 28-foot 5-inch spans and bents 2, 3 and 4	54 545
28-foot 5-inch span	11 215
108-foot truss span	124 725
27-foot span	14 020
27-foot span and bents 5 and 6	35 385
27-foot 2-inch span	12 340
Carried forward	290 415

		Pounds.
Brought forward		290 415
162-foot truss span		202 010
Shore arms of cantilever		1 376 978
River arms of cantilever		691 360
Suspended span		335 185
Two 254-foot spans		809 160
55-foot girder spans		45 550
30-foot spans and bents 7 and 8		37 396
31-foot 5-inch span		15 025
30-foot span and bents 9 and 10		33 566
31-foot 5-inch span		15 235
30-foot span and bents 11 and 12		33 396
31-foot 5-inch span		15 070
29-foot span and bents 13 and 14		28 432
50-foot girder		36 715
		3 965 483
Rivets, bolts, etc	31 790	
Hook bolts, nails and spikes	36 945	
Finials and cresting	17 075	
Name plates and castings	5 345	
Hand railing, Rail	176 695	
Braces	2 405	
Posts	12 955	
Lamps, posts and braces	3 590	
Newel posts	5 635	
Stairways	44 865	
		337 300
		4 302 783

Lumber (white oak) for floor.....551 434 feet B. M.
Paint, 650 gallons (two coats).....1 gallon for $3\frac{1}{3}$ tons.

Erection of Central Bridge.—About the middle of September, 1890, although the masonry was in somewhat of a sad plight, and it was clearly evident that the contractors could not complete their work as agreed upon, there was sufficient work done to enable hopes to be formed that the work could be completed somewhere in the neighborhood of February or March, 1891.

On the Cincinnati end nothing could be done owing to the right of way not yet being clear and the superstructure not yet completed. Work was therefore begun on the Newport end. In the early part of October the viaduct on the Newport side was started and was completed by the 25th of the month. On October 30th the girders across Front Street were placed in position. On November 12th the raising of the iron span between Piers 8 and 9 was begun, the false work having been placed between these piers during the latter part of October, and by the 15th the spans were connected. On the 24th it was fully riveted up and the stringers and floor beams were in place. On November 25th the false work for the span between Piers 7 and 8 was started and was finished December 3d, and by December 9th this span was fully coupled up.

The question as to whether the Newport shore arm of the cantilever between Piers 6 and 7 should be erected at this time now arose; the work on the caisson for Pier No. 5 was progressing favorably, and indications were that it would be prosecuted to a finish without interruption. The river looked very favorable (water stage 15 feet), and it was decided to erect this arm. On December 14th the work on the false work was started and was finished by the 20th, and on the 21st the raising of iron began. On the 22d reports from the head waters of the river indicated that considerable of a rise could be expected; so that additional energy was used, and by the 23d this span was coupled up without the floor beams or stringers.

It was calculated that the trusses could stand their own weight alone, if found necessary to remove the false work before the river arms of the cantilever were erected. As the indications were that the river would reach a stage of over 30 feet, in which false work could not be held, it was determined to take it out, and this was done by the 28th of the month. The work on the Newport side was then stopped until some reasonable assurance of the completion of Pier No. 5 could be had, and the work on the erection of the river arm was not started until the following May. When this work was begun the material for the cantilever arm was taken out in barges and hoisted into place. This work was very difficult and expensive, but it was not safe to take any material out on the shore arm without having false work to support it.

After extending the river arm three panels and hanging to it the

traveler weighing about 60 tons, permission was given the contractor for erection, to place the floor system of the shore arm in position and to carry out the material along the bridge. The erection towards the center of the cantilever was carried on slowly, as there was no occasion to push this until Pier No. 5 was completed. It, however, was finished June 18th, at the same time as Pier No. 5.

Prior to January 1st the right of way on the Cincinnati side had been fully obtained, cleared, and the substructure finished. On January 15th the Cincinnati viaduct was erected and fully riveted.* By March 11th the 162-foot span between Piers 3 and 4 was coupled up. On May 25th the false work on the Cincinnati shore arm was started and finished June 12th. July 3d this arm was completely coupled up, and by the 14th the inside traveler necessary to erect the cantilever was completed, and the work of erecting the Cincinnati cantilever arm started. On July 22d the two arms of the cantilever met and were coupled up without any difficulty. The wedges and screws used gave great satisfaction and enabled the span to be readily adjusted. On August 29th, 1891, the whole work was completed and opened for the use of the public.

THE CANTILEVER SPAN.

By F. C. Osborn, M. Am. Soc. C. E.

One of the objective points in the designing of this structure was the elimination, as far as possible, of undulatory and vibratory motion from passing loads. To this end the stringers were riveted rigidly to the floor beams, the floor beams in turn rigidly attached to posts and suspenders, and the latter made in compression form in order to better resist any tendency to vertical vibration. The lower lateral bracing is made of angles, arranged in a double triangular system, the angles attached to stringers at all intersections and to each other at center of panel; the attachment to chords is by means of wing plates directly to main truss pins.

* Plate XXXI.

ANCHORAGE ARM OF CANTILEVER ON CINCINNATI SIDE, JULY 14TH, 1891.

The portal bracing at the anchorage end of the shore arm is in box form, taking hold of both top and bottom flanges of the end post by means of large gusset plates and also attaching securely to the top and bottom flanges of the top chord. The portal rods are made double and attach to long pins passing through the gussets.

The top chord bars of the river cantilever arms are made in two-panel lengths, and are supported by the light vertical posts and top lateral struts in such a way as to clamp them securely in position and at the same time effectively transmit the wind pressure at the panel point to the top lateral bracing.

Owing to the sharp grade of the cantilever spans the proper position for the tall posts over the piers became an interesting question. If they were made vertical, the other posts and suspenders being normal to the bottom chord and grade, it would make a short panel on one side of the post and a long one on the other, and give the post the appearance of leaning up-grade, as well as an awkward look on account of not being parallel with the posts on either side of it. These objections could have been met, of course, by making all posts and suspenders vertical instead of normal to the grade. If this was done, however, it would have to be done on the adjacent spans, two of 254 feet each and one of 162 feet, and the extra expense would have been greater than was thought justifiable. After considerable study and the making of scale drawings of the several combinations, it was decided to make all posts, including the large ones over the pier, perpendicular to the grade.

The camber calculations for the river arm of the cantilever were made on the basis of the full dead load and one-half only of the maximum live load strain. The compression members were lengthened and the tension members shortened by an amount corresponding to their change in length from the strains caused by the above loading. The lengths of members in the shore arm were calculated as though the bottom chord was perfectly straight and the posts perpendicular to it, no allowance being made for upward or downward deflection. After swinging, and before the erection of the river arm, this span would deflect downward; under the dead load alone of the completed structure it would deflect upward; with the live load covering this arm alone it would deflect downward at the anchorage end and upward at the other, the bottom chord taking the form of a reverse curve.

The calculated maximum anchorage strain is 136 000 pounds at the

end of each truss, and is taken up by a 6 x 1¼-inch steel eye-bar passing 20 feet into the masonry and attached to a box girder 7 feet square and 2 feet deep.

Provision for alternate strains of tension and compression in the top chord of the anchorage arm is made by using eye-bars for the full tension strain and a built member for the full compression strain. The compression chord is prevented from taking up any tensile strain, by means of oblong pin holes which permit the compression members to separate at the joint.

APPENDIX I.

SPECIFICATIONS FOR SUBSTRUCTURE OF THE "CENTRAL BRIDGE" OVER THE OHIO RIVER, BETWEEN CINCINNATI AND NEWPORT, FOR THE CENTRAL RAILWAY AND BRIDGE COMPANY.

Abutments.—The abutments or ramps on the Cincinnati and Newport sides shall be respectively about 277 and 330 feet long; they shall be formed of two side walls of masonry capped, as shown on plans, with coping courses 12 inches thick and 2 feet wide, supporting the side railings, and form the remainder of the width of the sidewalks with Berea sandstone flags not less than 9 inches thick, laid in cement with parallel joints, and a front wall supporting the end of the iron superstructure, capped with limestone not less than 18 inches thick. The ends of the flagstone on the roadway side shall be supported on a good and suitable foundation, as the engineer may direct.

These walls shall be built of freestone ashlars not less than 10 inches thick; they shall be founded on a bed of concrete or on pile foundations, as the nature of the ground may require. The spaces between the walls shall be filled with earth, gravel or broken stone, or brick deposited in 12-inch layers, upon which the pavement and tracks for the wagon-ways and tramways shall be laid.

Pedestals.—There shall be twelve pedestals of masonry for the viaduct approach on the Cincinnati side, and sixteen for the viaduct approach on the Newport side. The pedestals shall be built of free-

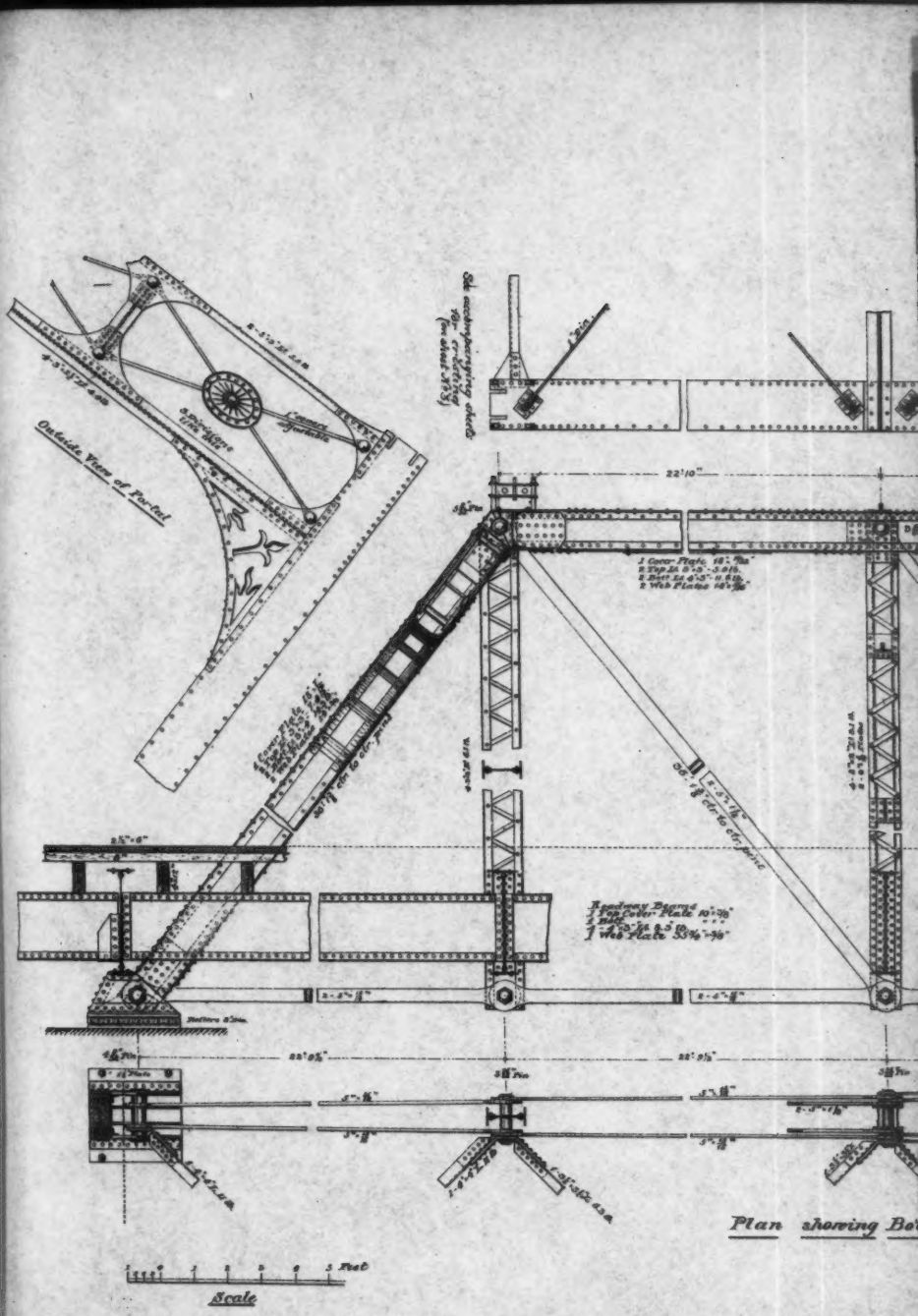
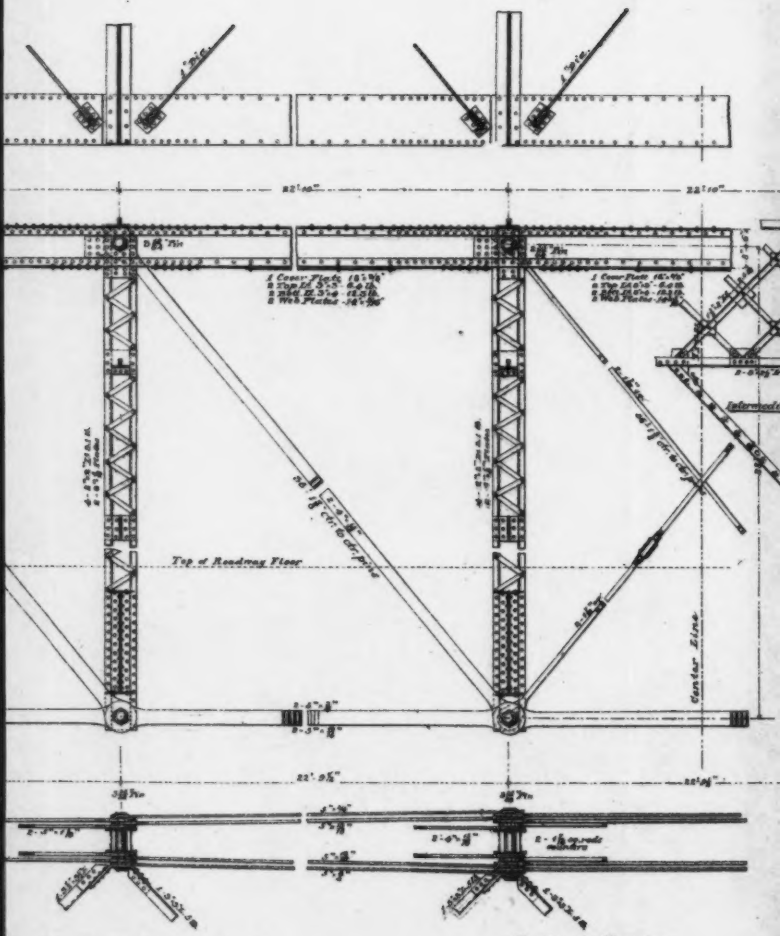

Outside View of Portal
Scale
Plan showing Bot

PLATE XX
TRANS.AM.SOC.CIV
VOL.XXVII.N9
KAUFMAN AND OS
CANTILEVER HIGHWA



showing Bottom Chord

Central Bridge
Cincinnati O. & Newport Ky.
Span 159'-6" c. to c. of end pins

Designed by
A. H. Porter.
Chief Engr King Br. Co.

Approved by
Ferris, Kaufman & Co.
Chief Engrs. Central B

LATE XXXII.
.SOC.CIV.ENGRS.
XXVII. Nº 545.
N AND OSBORN ON
E HIGHWAY BRIDGE.

Ky. —

ni & Co.
Central Br. Co.

Central Bridge

— between —

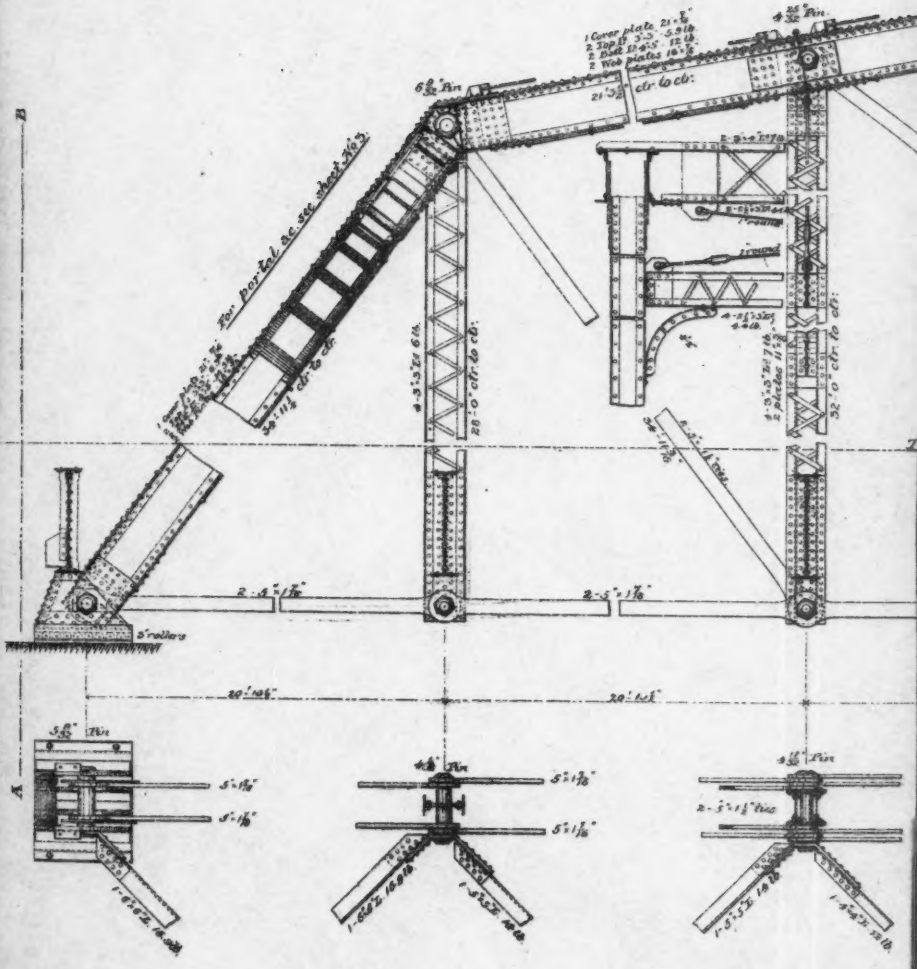
Cincinnati O. and Newport Ky.

Designed by
A. H. Porter
Chief Engr King Br. Co.

Approved by
Ferris, Kaufman & Co.
Engineers
Central Bridge Co.

— Notes. —

Span 250'6" c. to c end pins.
12 panels @ 20'-10½" (bottom)
Trusses 25'-11" ctr. to ctr.
2 Spans like this.



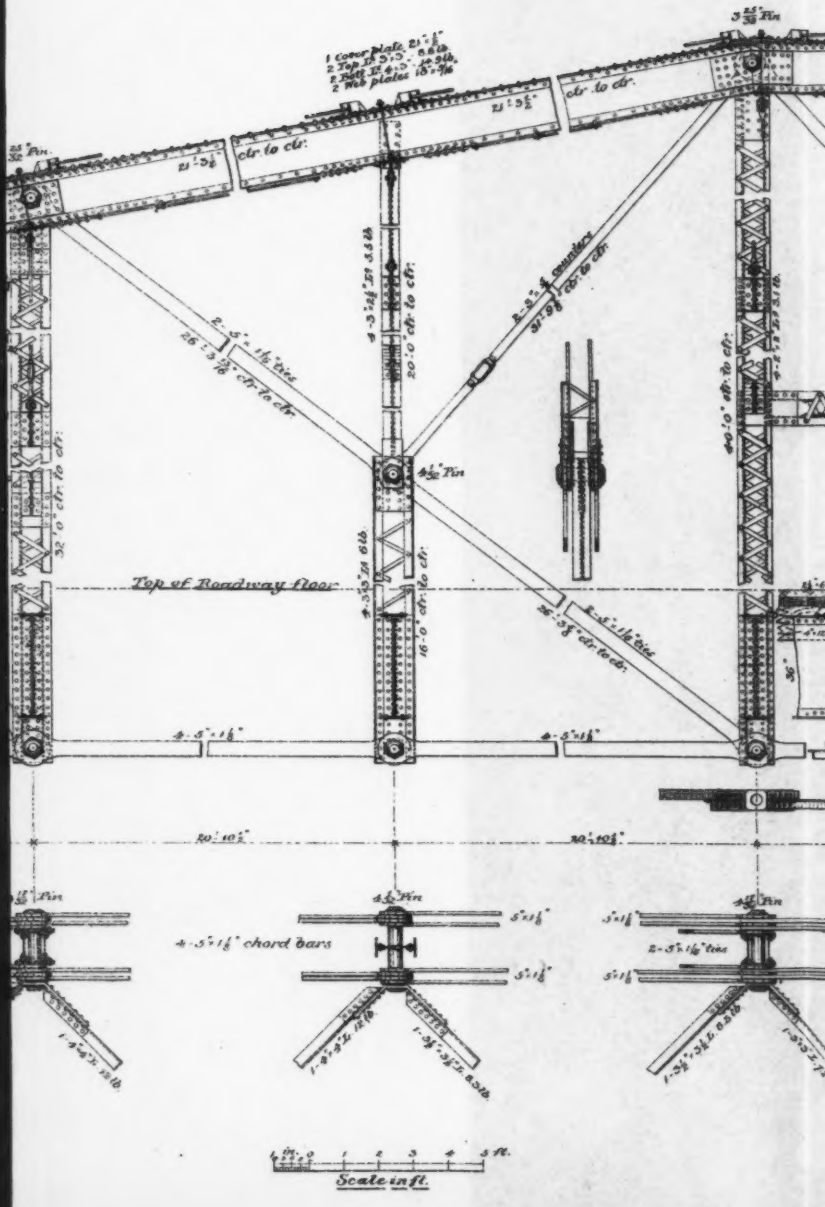
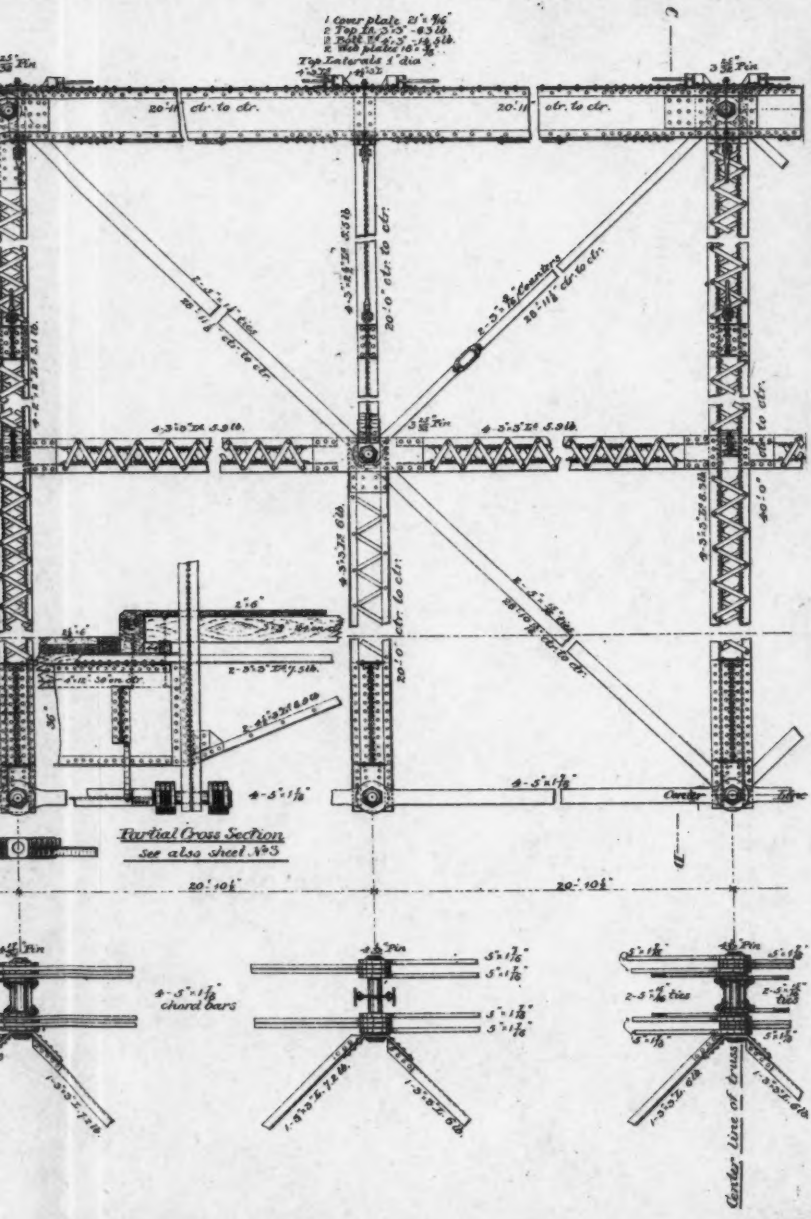
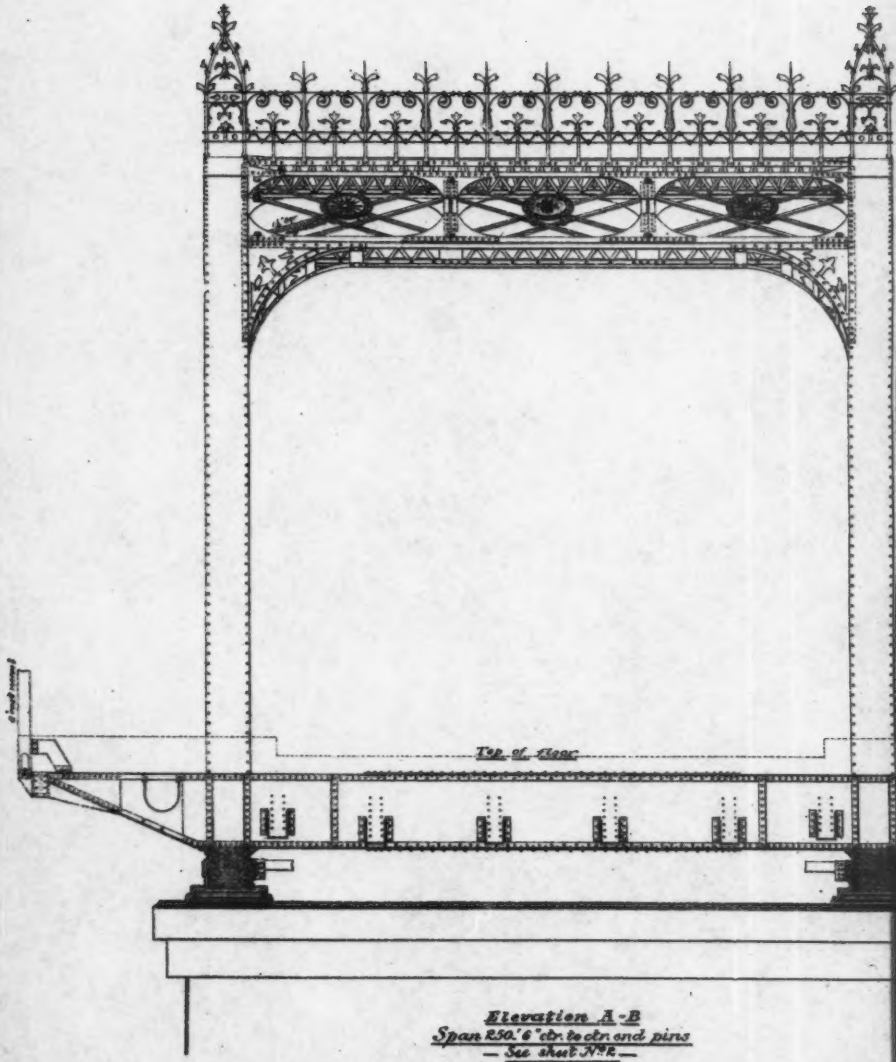

Top of Roadway floor
clr. to clr.
4-5"×1⅛" chord bars
Scale in ft.

PLATE XXXIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXVII. No. 545.
KAUFMAN AND OSBORN ON
CANTILEVER HIGHWAY BRIDGE.



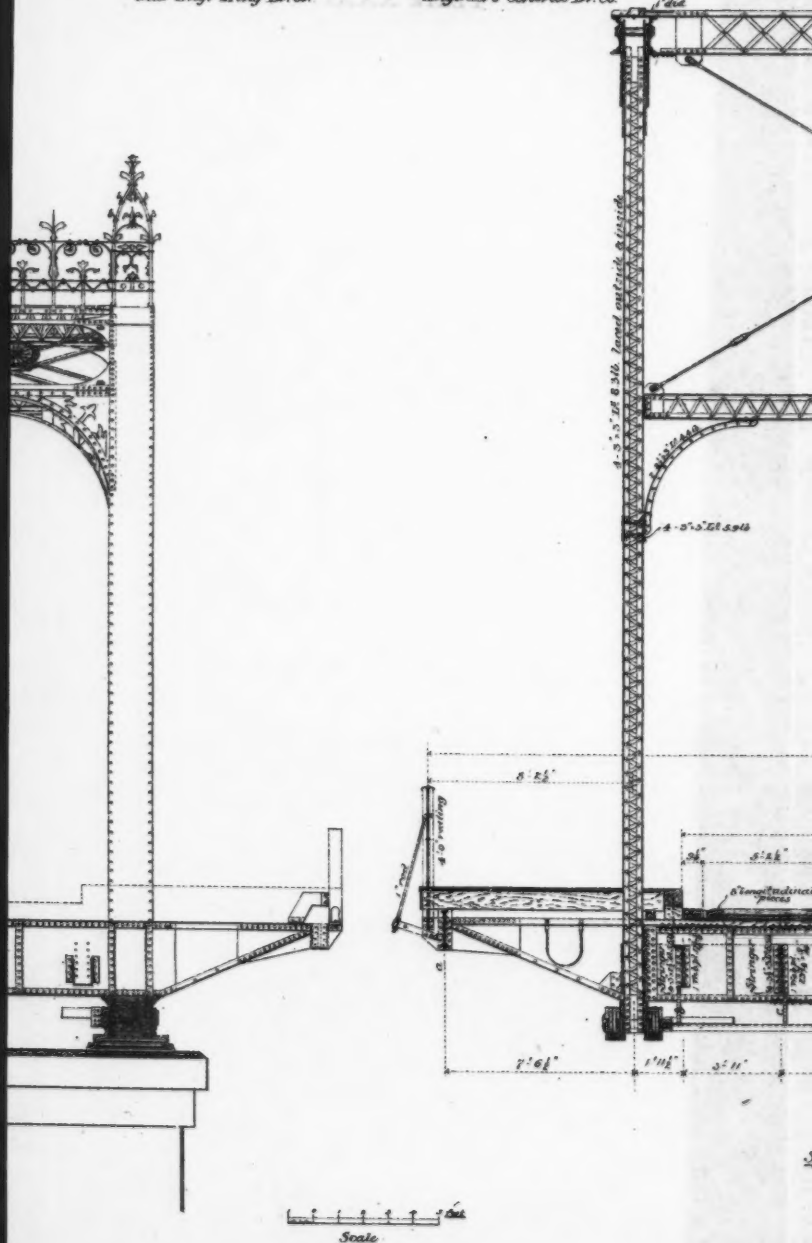


Elevation A-B
Span 250'6" ctr. to ctr. end pins
— See sheet No 8 —

Central Bridge
— between —
Cincinnati O. and Newport Ky.

Designed by
A. H. Porter
Chief Engr King Br. Co.

Approved by
Ferris, Kaufman & Co.
Engineers Central Br. Co.



Scale

PLATE XXXIV
TRANS. AM. SOC. CIV. ENGRS.
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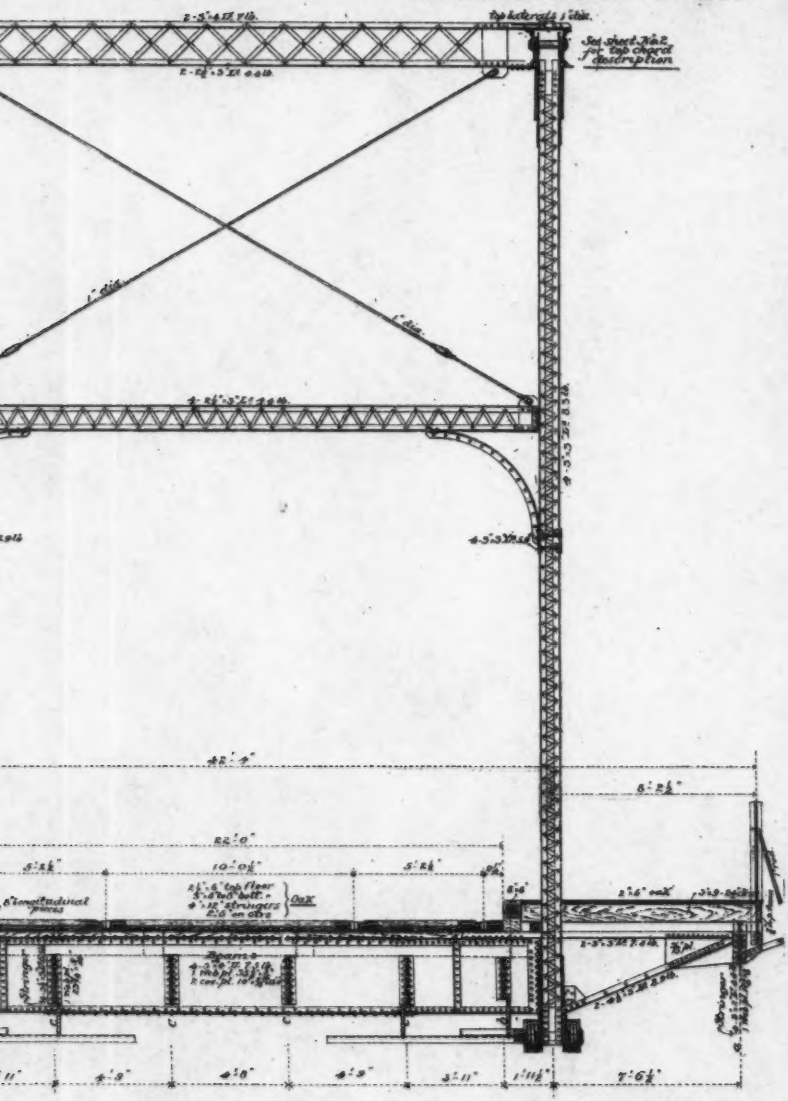
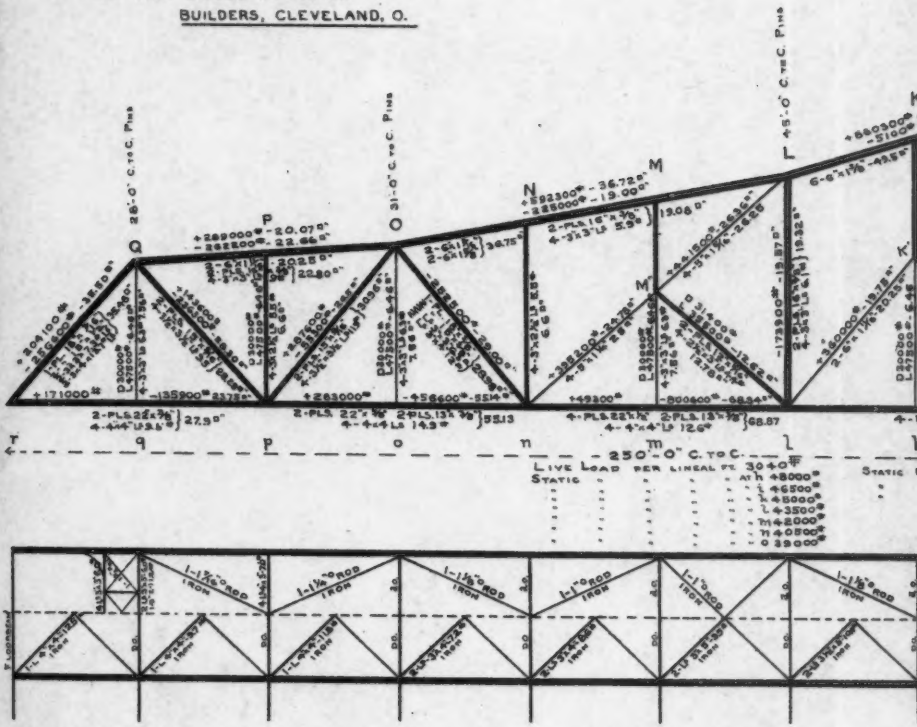
KAUFMAN AND OSBORN ON
CANTILEVER HIGHWAY BRIDGE.



E. Dol.

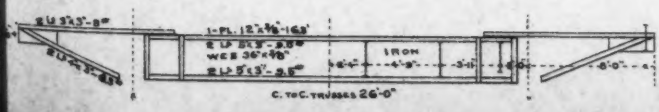
KING IRON BRIDGE & M'F'G. CO.
BUILDERS, CLEVELAND, O.

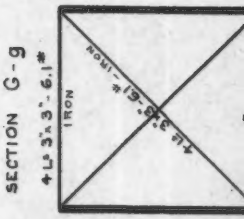


OUTSIDE ROADW'Y STRINGER
WEB 26"x 5/16" IRON
2 Lp 3 1/2"x 3"x 6.8#
2 Lp 3 1/2"x 3"x 6.8#

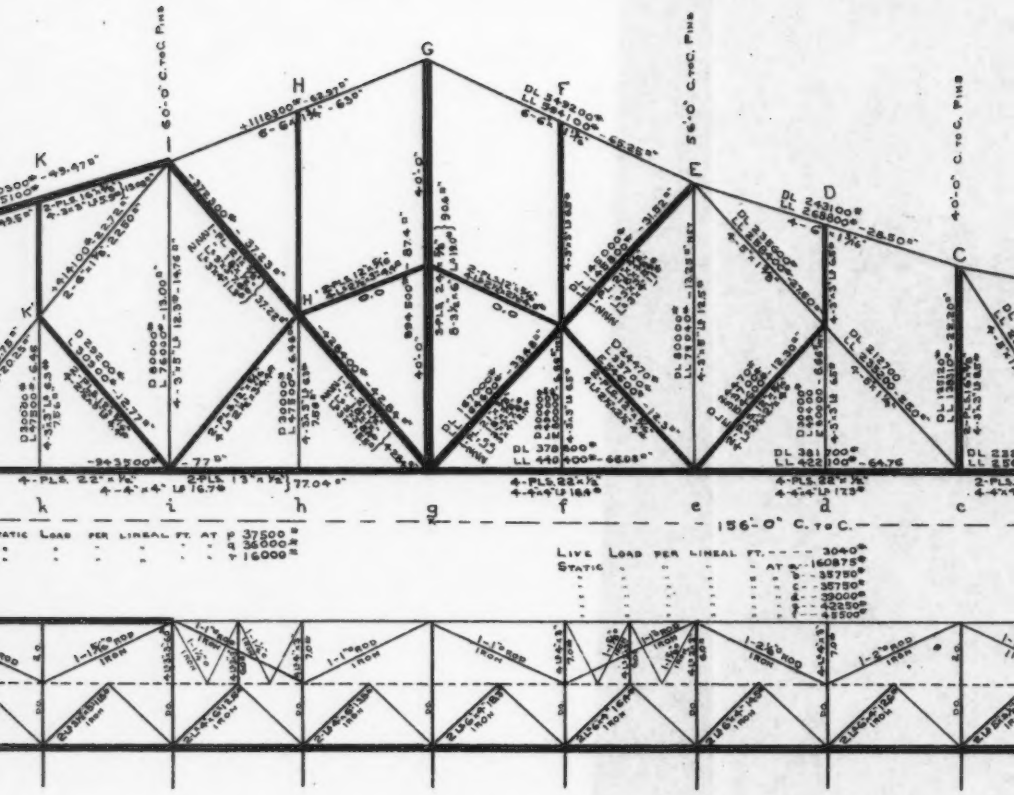
CENTER ROADWAY STRINGER
WEB 26"x 5/16" IRON
2 Lp 4"x 3"- 7.8#
2 Lp 4"x 3"- 7.8#

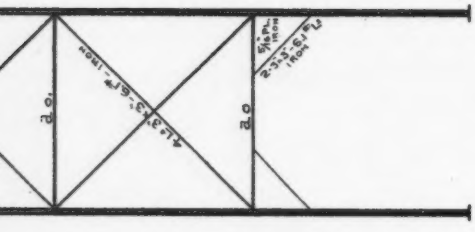
SIDEWALK STRINGER
WEB 22"x 1/4" IRON
2 Lp 3"x 2 1/2"x 5.9#
2 Lp 3"x 2 1/2"x 5.9#





ALL MATERIAL STEEL EXC





LIVE LOAD – For all truss members receiving more than one panel load, eighty (80) pounds per square foot for suspended span and cantilever arms. For stringers, floorbeams, and long suspenders one hundred (100) pounds per square foot of clear roadway and sidewalks, or an Aveling and Porter fifteen ton steam road roller.

In the calculation of stresses the following conditions of live load will be assumed.

For main truss members, the roadway and both sidewalks will be considered loaded

For long suspenders, roadway and one sidewalk only will be considered loaded, and for floorbeams, the roadway will be considered loaded with sidewalks unloaded, also roadway unloaded with sidewalks loaded.

The above 80 pounds per square foot is taken on clear roadway and sidewalks.

WIND
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EXCEPT WHERE OTHERWISE MARKED.

PLATE XXXV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXVII. No. 545.
KAUFMAN AND OSBORN. ON
CANTILEVER HIGHWAY BRIDGE.

Strain Sheet

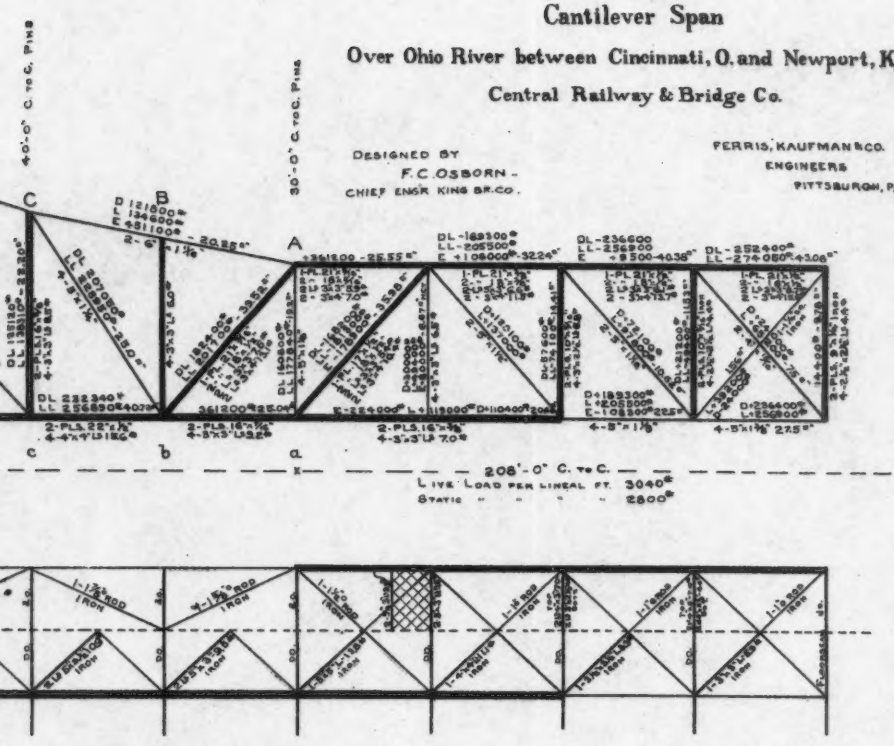
Cantilever Span

Over Ohio River between Cincinnati, O. and Newport, Ky.

Central Railway & Bridge Co.

DESIGNED BY
F. C. OSBORN -
CHIEF ENGR KING BR. CO.

FERRIS, KAUFMAN & CO.
ENGINEERS
PITTSBURGH, PA.



WIND LOAD - Stresses calculated for a pressure of thirty (30) pounds per square foot, on the exposed surfaces of both trusses and railings and a moving load surface of six (6) feet per lineal foot of bridge.

LATERAL STRUTS - Lateral struts will be proportioned to resist the resultant due to an initial stress of ten thousand (10,000) pounds per square inch upon all rods attached to them, when this is in excess of wind stress. The fiber stress due to weight of strut must be considered and be deducted from the unit stress specified.

Approved:
Ferris, Kaufman & Co.
By McKeefrey
May 31, 1890.

ALLOWED STRAINS PER SQ. INCH

TRUSSES

	Iron	Steel
Tension Members.		
Eyebars and counters	$10000\left(1+\frac{\text{min. stress}}{\text{max. stress}}\right)$;	$12000\left(1+\frac{\text{min. stress}}{\text{max. stress}}\right)$
Shapes and built angles (net sec)	10000 ;	12000
Lateral rods	18000	
Compression Members		
Square ends	$9000-30\frac{l}{r}$;	$15000-60\frac{l}{r}$
One square and one pin end	$9000-35\frac{l}{r}$;	$15000-70\frac{l}{r}$
Pin ends	$9000-40\frac{l}{r}$;	$15000-80\frac{l}{r}$
Lateral struts	$11000-50\frac{l}{r}$	

In which l = distance between supports in inches
r = least radius of gyration in inches

Alternate Tension and Compression

For compression only use compression formula

For greater stress $14000\left(-\frac{\text{min. stress}}{\text{max. stress}}\right)$; $12000\left(-\frac{\text{min. stress}}{\text{max. stress}}\right)$

Use the one giving the greater area of section.

Floorbeams and Stringers

Tension (net section) $10000\left(1+\frac{\text{min. stress}}{\text{max. stress}}\right)$; $12000\left(1+\frac{\text{min. stress}}{\text{max. stress}}\right)$

Compression - use same gross area as tension flange.

Note: Pin holes in ... than the true ...

— 0 —

Central Bridge
Cincinnati O., and Newport Ky.
Cantilever Arm

Designed by
F. C. Osborn
Chief Engr. King Bridge Co.

Approved by
Ferris, Kaufman & Co.
Engineers Central Bridge Co.

Sizes of material used in chords, diagonals, lowers, etc.

Name	Cover plates	Web plates	Top angles	Bott. angles	Eye bars	Notes
End post Q	1-18"×¾"	2-18"×¾"	2-3½"×3½"×¾"	2-5"×4"×7/16"		Steel
Upper chord Q	None	2-18"×1"	2-3½"×3½"×½"	2-5"×4"×½"	2-6"×1¼" counter	"
" " QR	"	2-18"×1"	2-3½"×3½"×½"	2-5"×4"×½"	2-6"×1¼" counter	"
" " R	"	2-18"×1"	2-3½"×3½"×½"	2-5"×4"×½"	2-6"×1¼" counter	"
" " RS	"	2-18"×1"	2-3½"×3½"×½"	2-5"×4"×½"	2-6"×1¼" counter 2-6"×1" main	"
" " S	"	2-18"×1"	2-3½"×3½"×½"	2-5"×4"×½"	2-6"×1¼" counter 2-6"×1" main	"
" " SN	"					"
Roll. - P L	"	½"×18"×1'-4"	2-6"×6"×⅝"	2-6"×6"×⅝"		"
" - U L	"	½"×18"×1'-4" 2 cover plates ½"×18"×1'-4"	" " "	" " "		"
" - L n	"	½"×18"×1'-4" 2 cover plates ½"×18"×1'-4"	2-6"×6"×⅝"	2-6"×6"×⅝"		"
" - n p	"	½"×18"×1'-4" 2 cover plates ½"×18"×1'-4"	2-6"×6"×⅝"	2-6"×6"×⅝"		"
" - p r	"	2-18"×⅝"	2-6"×6"×⅝"	2-6"×6"×⅝"		"
Diagonal Q R	"	2-18"×⅝"	2-6"×6"×½"	2-6"×6"×½"		Cast iron
" " R L	"	" " "	" " "	" " "		"
Floor beams P P'	1-21"×1"	2-18"×1"	2-3½"×3½"×½"	2-3½"×3½"×½"		Steel
" L K	None	2-18"×⅝"	2-2½"×2½"×½"	2-2½"×2½"×½"		Cast iron
" L'K'	"	2-18"×⅝"	2-2½"×2½"×½"	2-2½"×2½"×½"		" "
" K O	"	2-18"×⅝"	2-3½"×3½"×½"	2-3½"×3½"×½"		Steel
" O P	"	2-18"×⅝"	2-3½"×3½"×½"	2-3½"×3½"×½"		"
" P K	"	2-18"×⅝"	2-3½"×3½"×½"	2-3½"×3½"×½"		"
Stringers		2-20"×½"	4-3½"×3½"×½" channels	4-3½"×3½"×½" channels		"
Floor beams	Bot. plate 21"×⅝"	1-32"×½"	1-3½"×3½"×½"	1-3½"×3½"×½"		Cast iron
Sidewalk brackets						
Brackets	None	1-22"×⅝"	2-3½"×3½"×½"	2-3½"×3½"×½"		" "
Outside cir.	"	1-22"×⅝"	2-3½"×3½"×½"	2-3½"×3½"×½"		" "
Side walk abutments	"	1-22"×½"	2-2½"×2½"×½"	2-2½"×2½"×½"		" "

Single load with 2"×⅝" iron bars top and bottom

Single lo... bottom

Center — Line

Sections 12-12 / 12-12

Section 13-13 — Section 14-14 — Section 15-15

All 2'-2"×2'-2"×.5 th top & bott plates & webs ½"

Plan — Section

— Anchor boxes —
7 feet square filled with concrete

Top laterals ... by means of ...

Top of roadway floor

Struts 4-5-4 27 th

Struts 4-5-4 27 th connected to pins inside posts

Sub struts 2-5-4 27 th

Struts 2-5-4 27 th

Struts 2-5-2 27 th

Bolt laterals 2-5-4 27 th

Bolt laterals 1-4-4 27 th

Bolt laterals 2-5-4 27 th

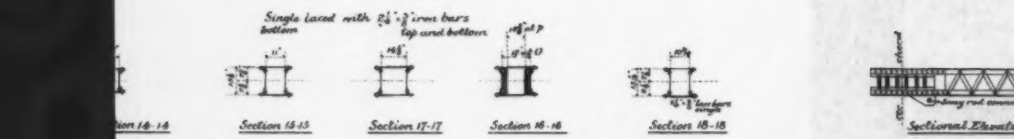
Bolt laterals 1-4-4 27 th

Struts 2-5-2 27 th

Struts 2-5-4 27 th

Struts 2-5

Note: Pin holes in web plates of top chord are made $\frac{1}{8}$" greater (on one side of hole only) than the true centre, to prevent compression members to take up tension and vice versa.



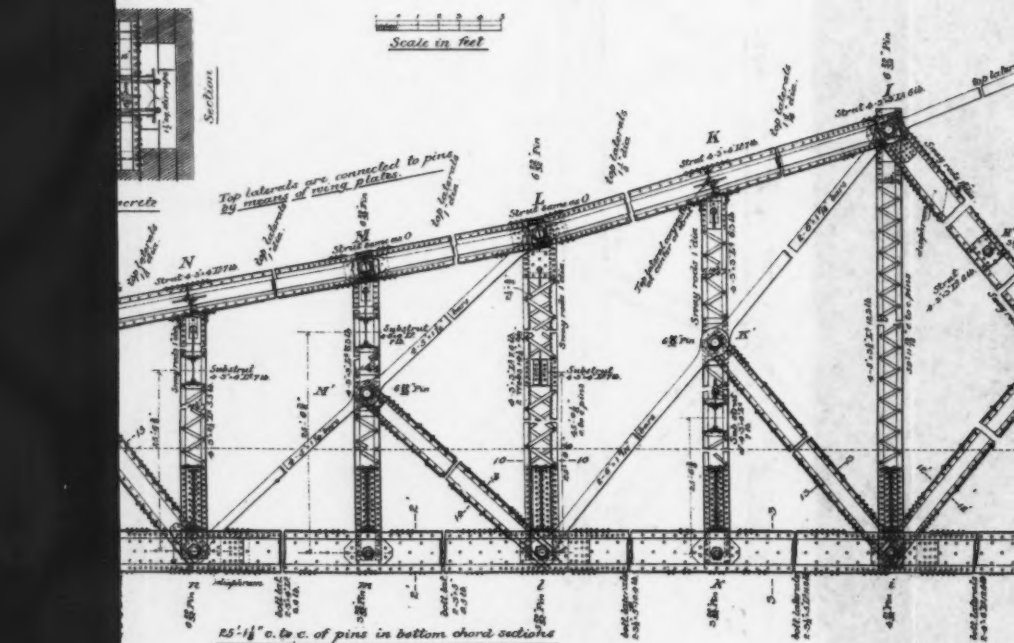
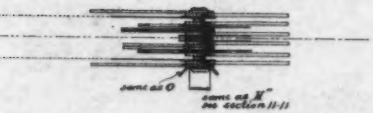
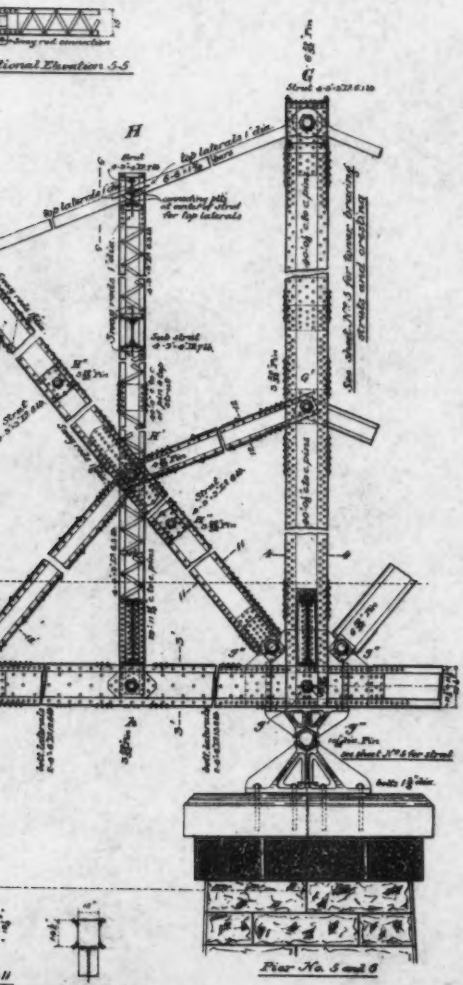

Scale in feet
Section
Top laterals are connected to pins by means of wing plates.
Strut same as O
Strut 4-5-4 27 lbs
Strut 4-5-4 27 lbs
Strut same as O
Strut 4-5-4 27 lbs
Strut 4-5-4 27 lbs
Strut 4-5-4 27 lbs

PLATE XXXVI.
TRANS.AM.SOC.CIV.ENGRS.
VOL.XXVII.No 545.
KAUFMAN AND OSBORN ON
CANTILEVER HIGHWAY BRIDGE.

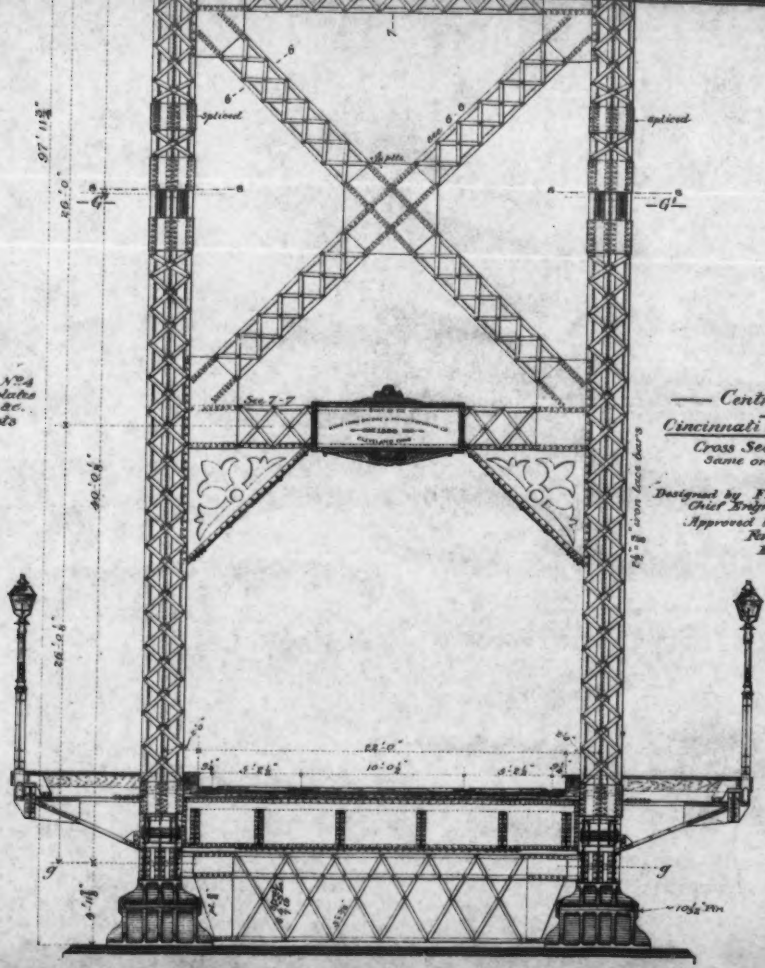
— I —





Section 10-10

B. Ind.



Note: See sheet No 4 for sizes of plates and angles &c. for main posts

— Central Bridge —
— betw. —
Cincinnati O. & Newport Ky.
Cross Section on Pier 5
Same on Pier No. 6

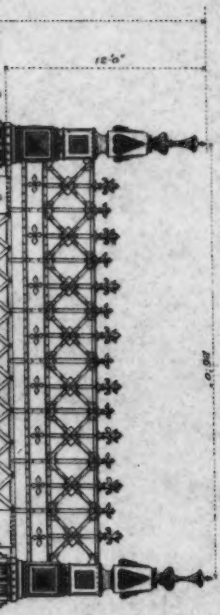
Designed by F. C. Osborn
Chief Engr King Bridge Co.
Approved by
Ferris, Hautman & Co.
Engineers Central Br. Co.

Scale

E. mt.

PLATE XX
TRANS.AM.SOC.C
VOL.XXVII.
KAUFMAN AND C
CANTILEVER HIGH

LATE XXXVII.
AM.SOC.CIV.ENGRS.
L.XXVII.Nº 545.
AN AND OSBORN ON
ER HIGHWAY BRIDGE.



Central Bridge
between
Cincinnati O. & Newport Ky.

Designed by
F C Osborn
Chief Engr King Br Co.

Approved by
Ferris, Kaufman & Co
Engineers Central Br Co.

River Arm of Suspended Span
520-0" c to c piers 5 & 6.

Scale of feet.

Bottom Laterals (area)
See also other sheets for
Chord Forms

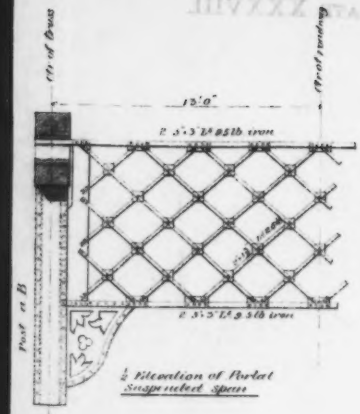
gf	0 6 x 5/16	19.38 lb	River arm
fe	0 6 x 5/16	19.38	"
ed	0 6 x 5/16	19.38	"
dc	0 6 x 5/16	19.38	"
cb	0 5 x 5/16	16.04 lb	"
ba	0 5 x 5/16	16.04	"
ab	1 3 x 5/16	17.86 lb	Suspended span
bc	1 4 x 5/16	21.41 lb	"
cd	1 3½ x 5/16	22.32 lb	"
de	1 3 x 5/16	23.21 lb	"

Pier No 5 and 6

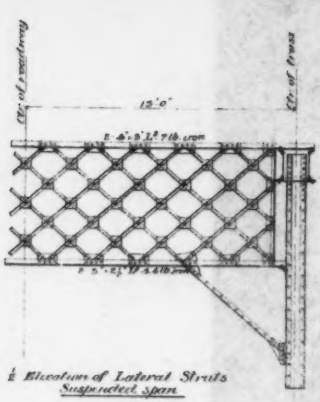
Section 1-1 gfe

Section 2-2 edc

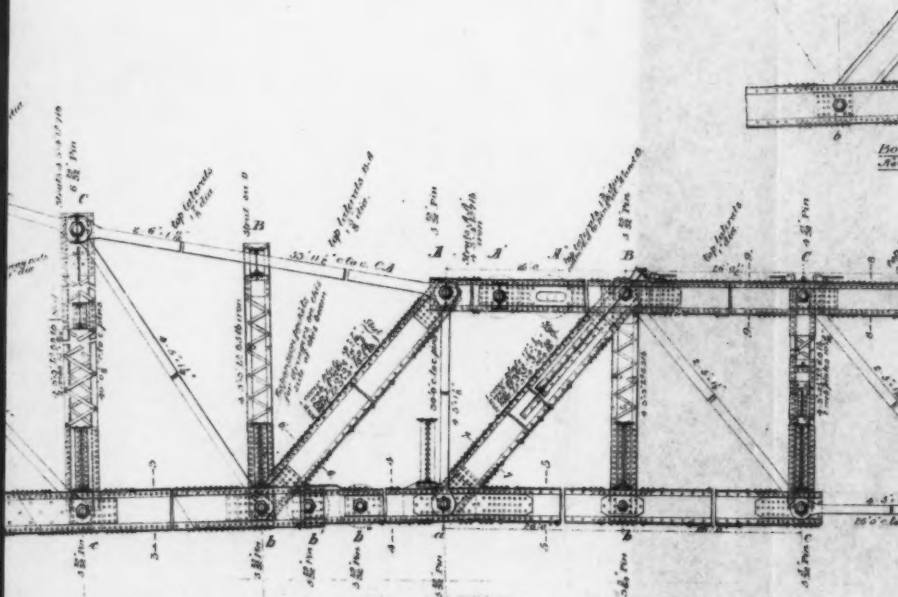
Section 3-3 cbb'

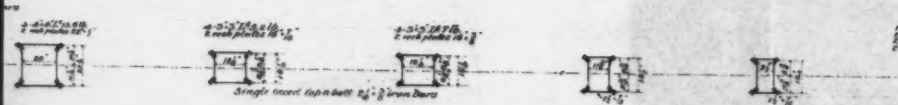


½ Elevation of Portal
Suspended Span



½ Elevation of Lateral Struts
Suspended Span





Section 3-3 cbb'

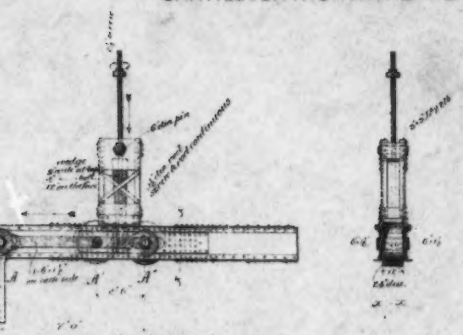
Section 4-4 ba

Section 5-5 abc

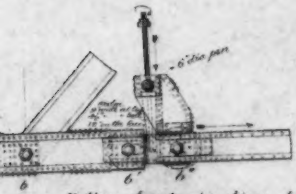
Section 6-6 bA

Section 7-7 aB

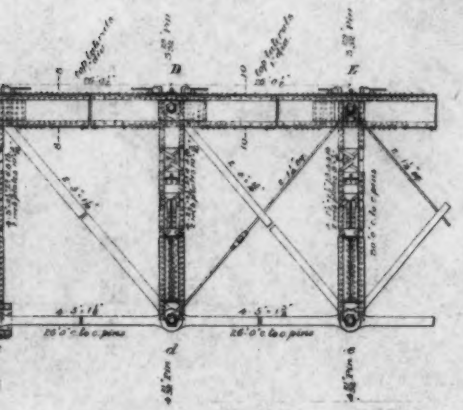
PLATE XXXVIII.
TRANS.AM.SOC.CIV.ENGRS.
VOL.XXVII.Nº 545.
KAUFMAN AND OSBORN ON
CANTILEVER HIGHWAY BRIDGE.



Top chord adjusting device.
Note. When adjustment is complete the short eye bars are taken away entirely, the pin at B is transferred to A' which is also inserted and pin also taken out lateral rods. A is provided with 6in expansion slot 18" long.



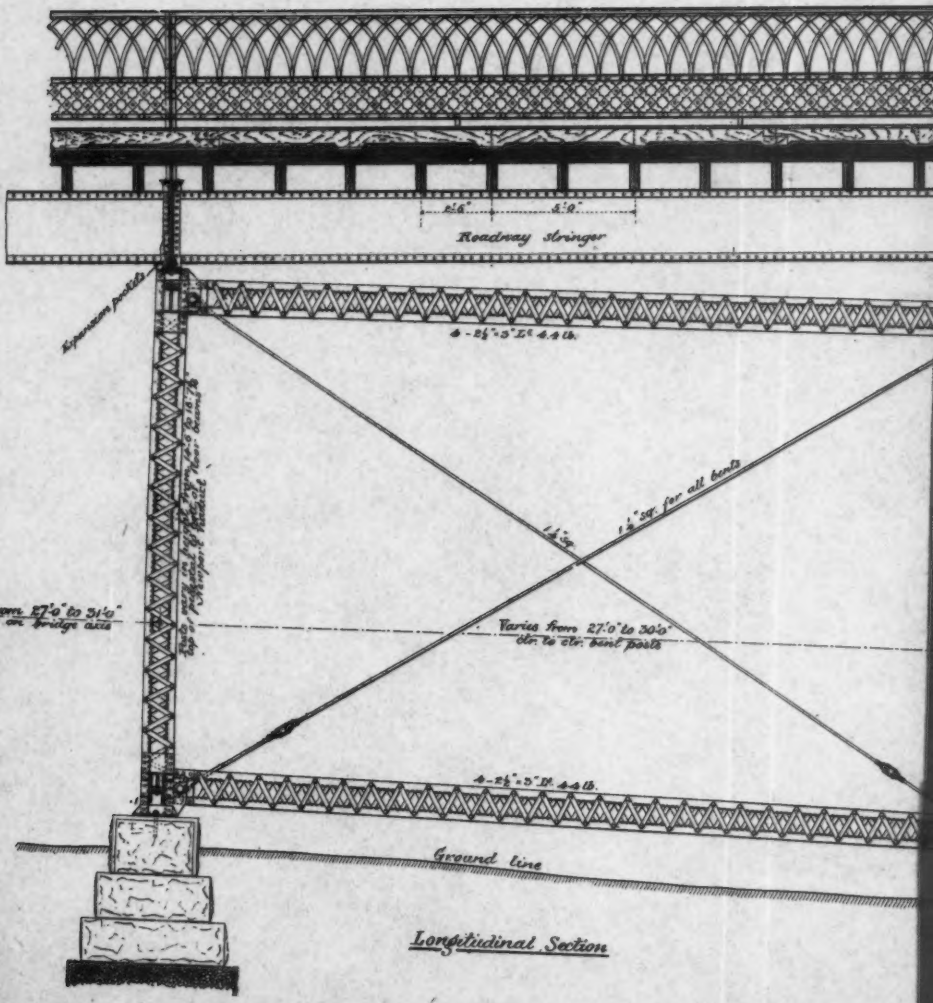
Bottom chord adjusting device.
Note. 18" expansion slot in small chord section at b'



Single laced at bottom of 1½" eye bars

Section 8-8 CD | Section 9-9 BC | Section 10-10 DE

B. Del.



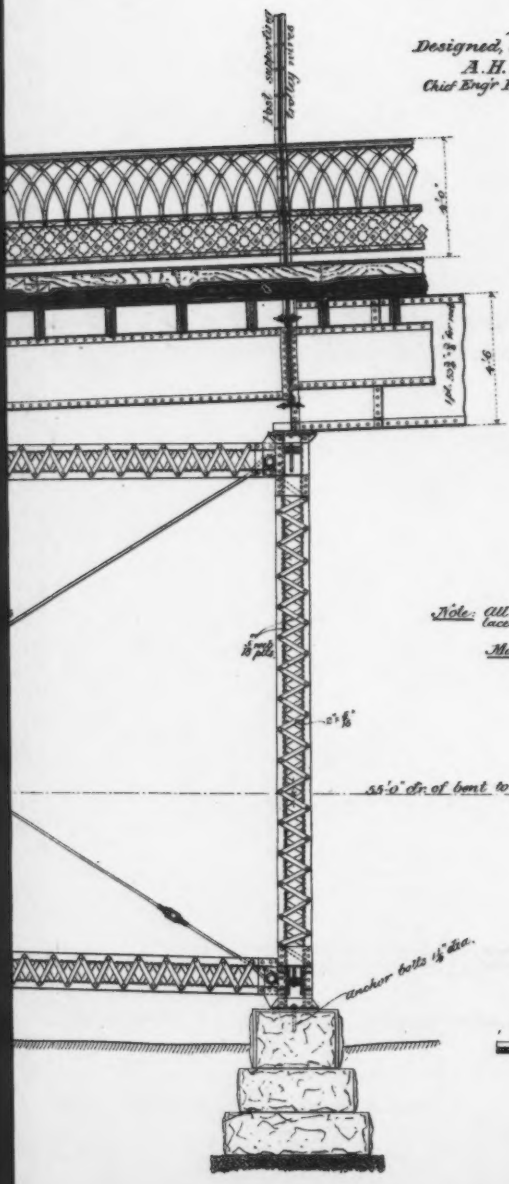
Longitudinal Section

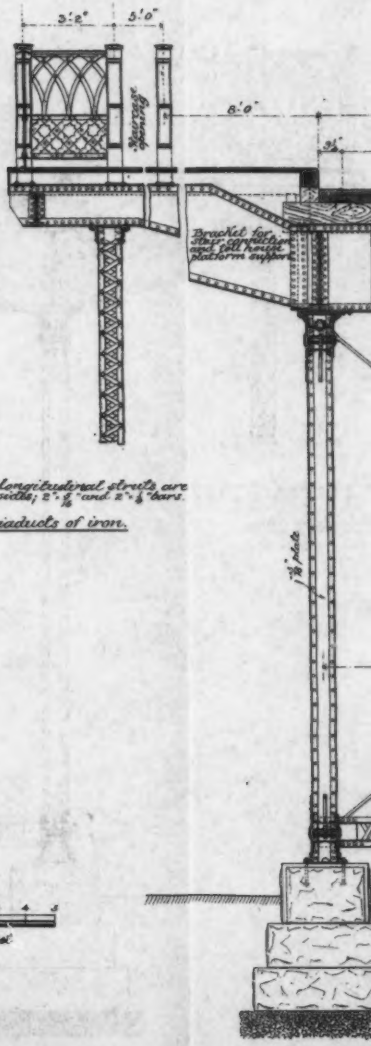
PLATE XXXIX.

Central Bridge
– between –
Cincinnati O. & Newport Ky.
Sections showing bents &c.

Designed by
A. H. Porter
Chief Engr King Br. Co.

Approved by
Ferris, Kaufman & Co.
Engineers Central Bridge Co.



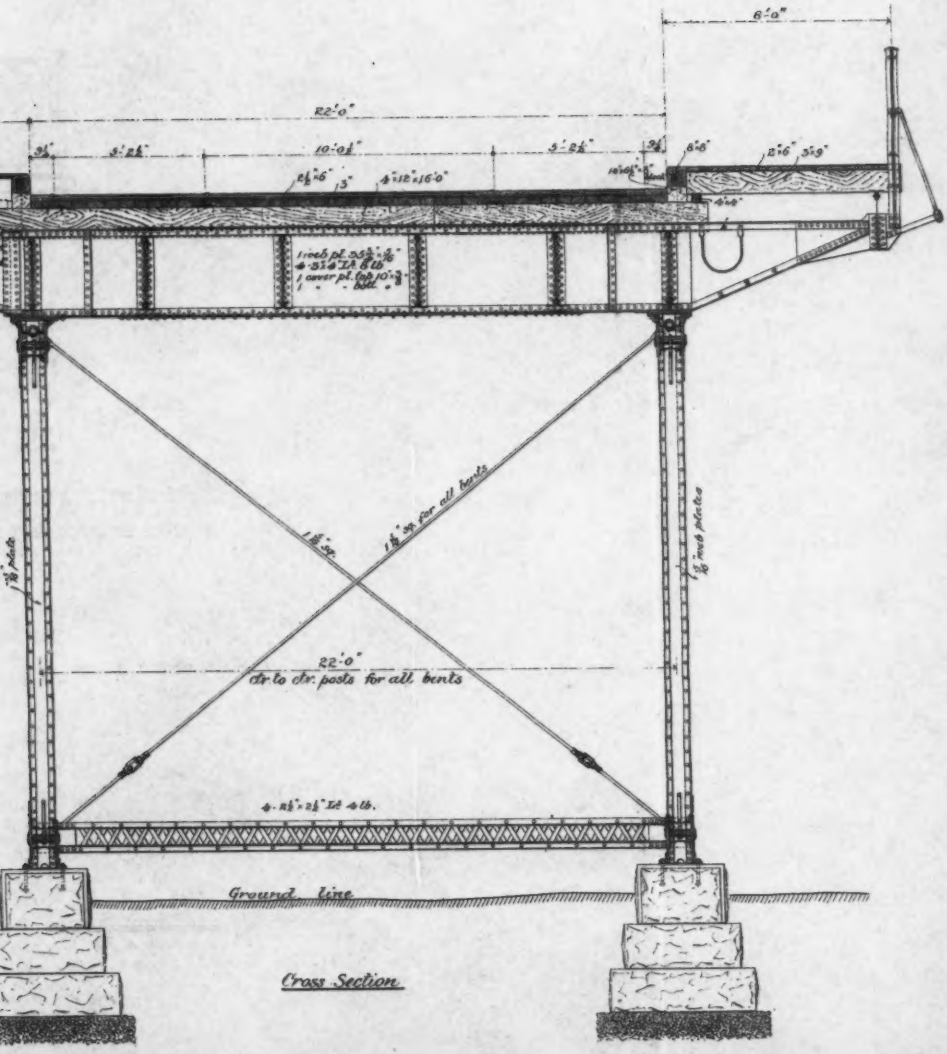


Note: All lateral and longitudinal struts are laced on all four sides; 2" × ⅜" and 2" × ¼" bars.

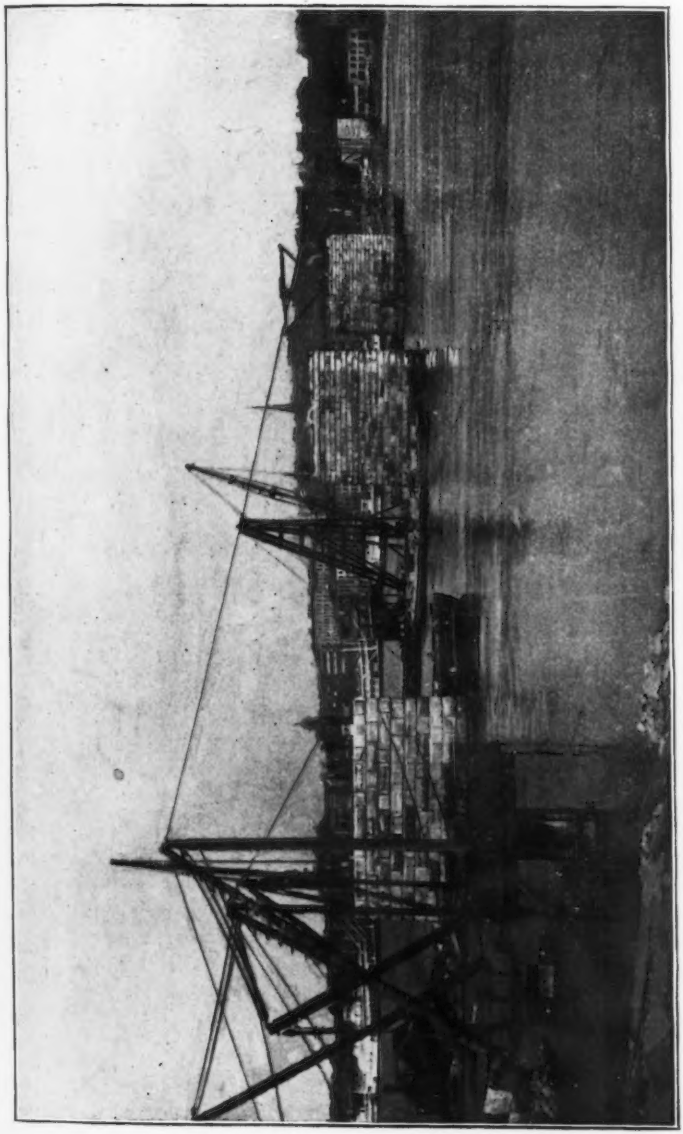
Material for viaducts of iron.

Scale of Feet

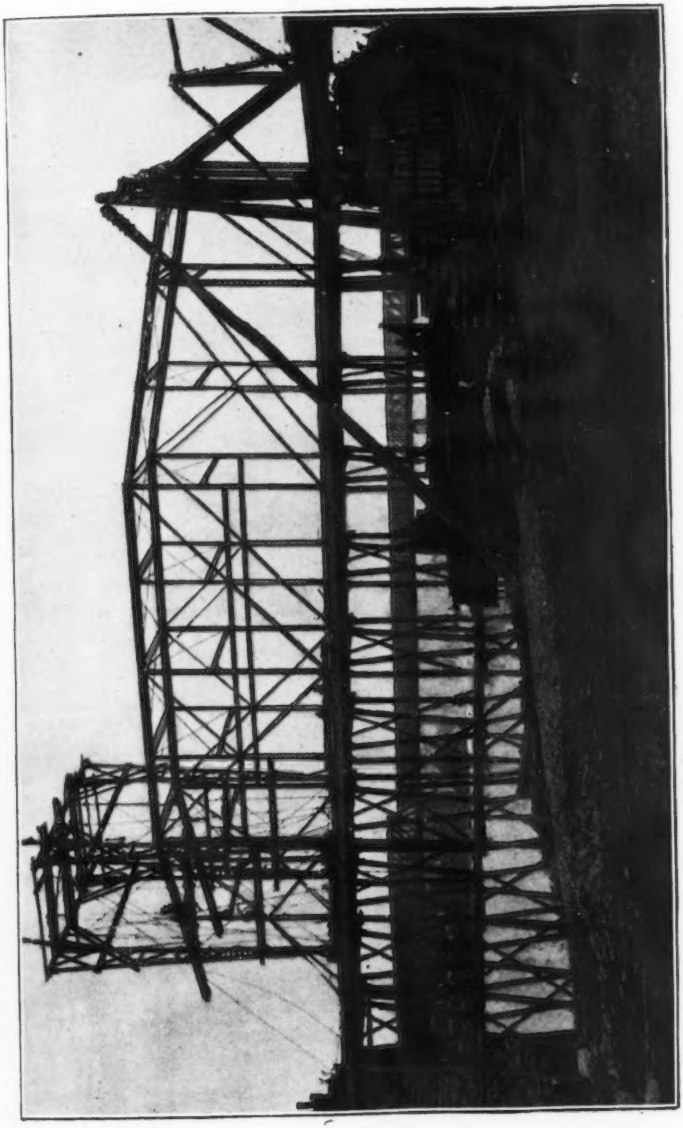
PLATE XXXIX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. XXVII. No 545.
KAUFMAN AND OSBORN ON
CANTILEVER HIGHWAY BRIDGE.



Cross Section.

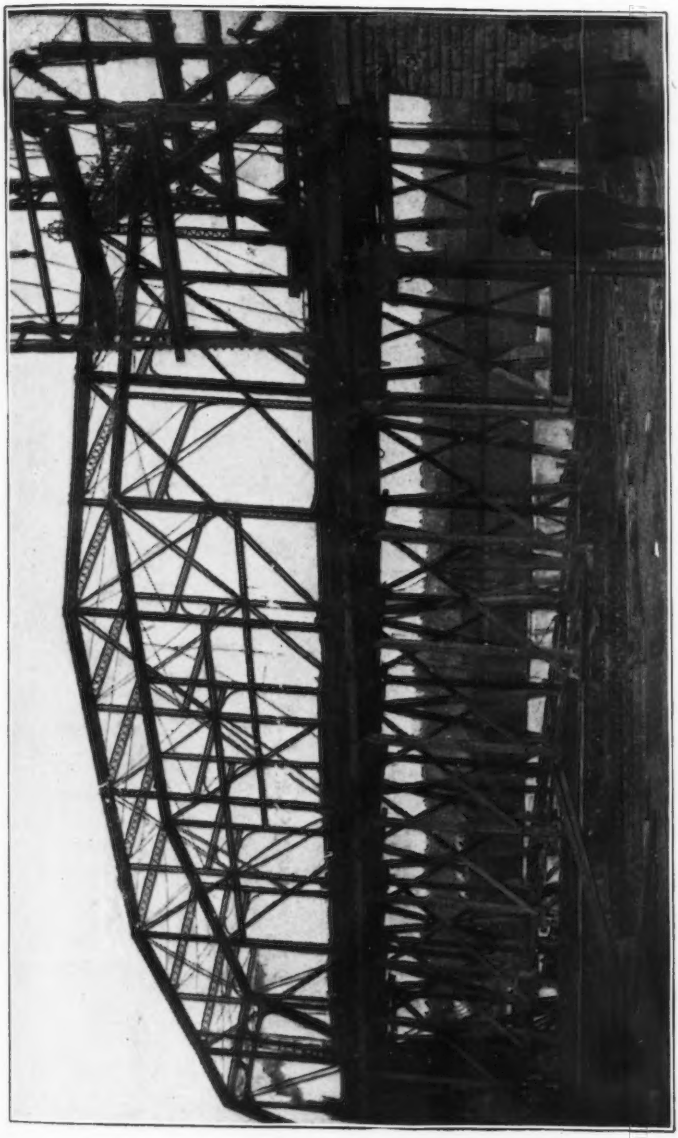


GENERAL VIEW OF RIVER PIERS, SEPTEMBER 20TH, 1890.

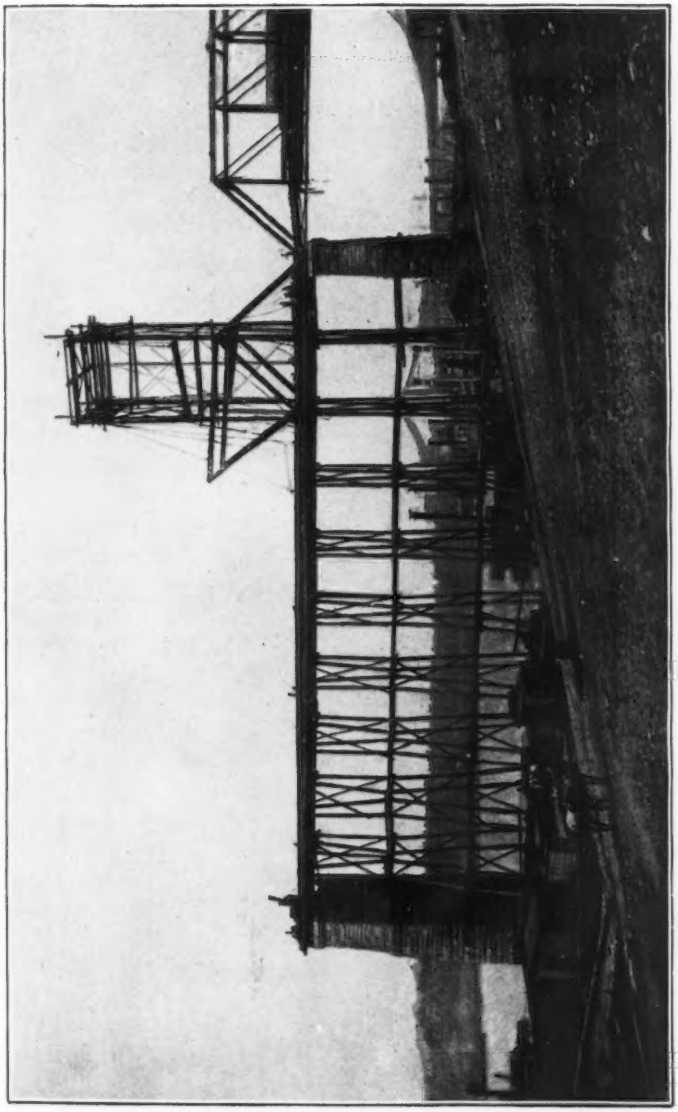


SPAN 8 AND 9, NEWPORT SIDE, NOVEMBER 14TH, 1890.

PLATE XLII.
TRANS. AM. SOC. CIV. ENGS.
VOL. XXVII, No. 545.
KAUFMAN & OSBORN ON CANTILEVER
HIGHWAY BRIDGE.



SPAN 8 AND 9, NEWPORT SIDE, NOVEMBER 17TH, 1890.



ANCHORAGE ARM OF CANTILEVER, CINCINNATI SIDE, JUNE 30TH, 1891.

stone ashlars not less than 18 inches thick, capped with a single block of limestone not less than 18 inches thick. They shall be founded on a bed of concrete, or on a pile foundation, as the nature of the ground may require.

Piers.—Numbering from the Cincinnati side, Piers Nos. 1, 2, 3, 4 and 9 shall be founded on pile or concrete foundations, the bottom of the masonry being from 6 to 10 feet below the surface of the ground.

Piers Nos. 5, 6, 7 and 8 shall be founded on the bed rock of the river. The rock shall be excavated from 4 to 6 feet in depth, and properly dressed to receive the first course of masonry; the spaces between the side walls of the pit and the masonry of the piers shall be filled with concrete to an even elevation with the bottom of the river.

Piers Nos. 1, 2, 3 and 9 shall be rectangular in shape, in a horizontal section, with a batter of one-half inch to the foot on all faces.

Piers Nos. 4, 5, 6, 7 and 8 shall also be rectangular in shape from the top of coping down to the elevation of high water. From high water down they shall have a semi-circular nosing at each end, as shown on plans.

The general dimensions for each pier shall be approximately as follows:

	Coping.	Heights.	Total.
Pier 1,	8 x 32	.. ..	23 feet.
" 2,	8 x 32	.. ..	34 "
" 3,	8 x 32	.. ..	42 "
" 4,	11 x 36	29 41	70 "
" 5,	12 x 36	34 79	113 "
" 6,	12 x 36	34 71	105 "
" 7,	11 x 36	29 70	99 "
" 8,	9 x 32	21 70	91 "
" 9,	9 x 32	.. ..	36 "

The masonry of Piers Nos. 1, 2, 3 and 9 shall be of freestone ashlars not less than 16 inches thick; the coping shall be no less than 18 inches thick, and of approved stone.

The masonry of Piers Nos. 4 and 8 shall be of Berea sandstone ashlars for the face, with concrete hearting below high water.

Piers Nos. 5, 6 and 7 shall be of Berea sandstone facing and Ohio River freestone hearting below high water.

The masonry of Piers Nos. 4, 5, 6, 7 and 8 shall be composed entirely of freestone ashlars above high water to the under side of cop-

ing. The depth of the courses shall not be less than 16 inches. The coping courses shall be of limestone of approved quality not less than 24 inches thick for Piers Nos. 5 and 6, and 18 inches thick for piers Nos. 4, 7 and 8.

Each pier shall have one or more footing courses. The anchorage for the ends of the Cantilever spans on Piers Nos. 4 and 7, as well as the anchor bolts in pedestals of the viaducts, shall be put in by the contractor for the substructure; the iron work for the same shall be furnished by the Bridge Company.

General.

All coffer-dams and scaffoldings used for the construction of the piers, as well as all surplus material excavated for the foundations of the same, shall be removed by the contractor before payment of the final estimate. The material excavated from the foundations of the river piers shall be deposited in such a place as not to cause any obstruction in any portion of the river. The place of deposit shall be satisfactory to the proper authorities.

Piers and pedestals shall be built of first-class masonry. The abutments shall be of second-class masonry. All joints of the masonry shall be neatly pointed off with rich cement mortar. No masonry shall be laid in freezing weather without permission of the Engineer.

All materials shall be inspected, and shall be used only when approved and accepted by the engineer. All work shall be done under the direction and to the acceptance of the engineer. All defective work shall be promptly taken down by the contractor on orders from the engineer, and rebuilt properly at the contractor's expense.

In the absence of the contractor from any part of the work, the engineer shall give his orders respecting that work to whomsoever it may be in charge of, or executing the said work, and his orders shall be respected and obeyed. The contractor assumes all risks arising from the weather, accidents or causualties of any kind.

Masonry details shall be prepared by the engineer for each structure, and a copy of the same shall be furnished the contractor before beginning the work.

Masonry shall be divided into two classes: first and second-class masonry.

First-class Masonry.—For Piers Nos. 4 and 8: The thickness of the courses shall vary from 16 inches thick to 30 inches thick, and the courses shall decrease uniformly in thickness from the bottom to the top of piers. The piers shall consist of alternate courses of headers and stretchers. They shall not be less than 3½ feet nor more than 6 feet long, and shall be no less than 16 inches nor more than 30 inches thick, nor less in width than one and one-quarter times the depth of the course to which they belong.

The casings shall be of Berea sandstone and the piers shall be filled with concrete made of Louisville selected and inspected cement. The casings shall be laid in alternate courses of headers and stretchers; the face stones shall be square; the joints shall be three-eighths of an inch in thickness. The vertical joints three-eighths of an inch in thickness shall extend backward from the face of the wall no less than 12 inches, and as much more as the stone will admit. The concrete filling shall be placed in the pier upon the completion of each course of the casing. It shall be mixed and deposited in place as specified under the head of "concrete."

All face stone must hold their size back in the heart of the wall that they show on the face.

All stone must lie on their natural quarry bed, and must be cleaned carefully and dampened before setting. They must have their beds and joints well dressed, and true to the proper plane. The beds shall be made as large as the stones will admit of. All face stones shall break joints not less than 12 inches.

No hammering on the stone will be allowed after it is set, but small inequalities may be pointed off carefully.

The masonry shall be rock faced, with no projections of more than 3 inches from the proper plane.

The belting and coping courses, as well as quoins, shall have drafts 1½ inches wide. The coping stones shall have parallel joints dressed throughout. They shall be of such dimensions as may be required by the engineer. They shall be tied together with iron clamps made of seven-eighths inch square iron; they shall extend 9 inches within the edge of each stone and their points shall extend 4 inches into each stone; the clamps shall be set in lead.

First-class Masonry for Piers Nos. 5, 6 *and* 7.—The piers shall consist of headers and stretchers, and there shall be at least one header to

every three stretchers, or more frequently if necessary in the opinion of the engineer.

Headers and stretchers shall not be less than 3½ feet nor more than 7 feet long, according to thickness, nor less in width than one and a quarter times the depth of the course to which they belong.

The thickness of the courses shall not be less than 16 inches nor more than 30 inches, and they shall decrease uniformly from the bottom to the top of walls.

Face stones must hold the size back in the heart of the wall that they show on the face.

All stones must lie on their natural quarry bed and must be cleaned carefully and dampened before setting. They must have their beds and joints well dressed and true to their proper plane.

The beds shall be made as large as the stones will admit; the vertical joints of the face must be in contact at least 6 inches measured in from the face. The face stones shall break joints not less than 12 inches. The backing shall be of good-sized, well-shaped stones, laid so as to break joints, and thoroughly bond the work in all directions. The joints shall not be less than three-eighths of an inch nor more than five-eighths of an inch thick. There shall be no spaces larger than 6 inches between the backing stones; they shall be filled with small stones laid flush in cement mortar. The whole of the masonry shall be laid flush in cement mortar, so as to fill thoroughly all joints, beds and spaces between stones. To remove all doubts as to this point, each course shall also be grouted, if required by the engineer.

No hammering on the stone will be allowed after it is set, but small inequalities may be pointed off carefully.

The masonry shall be rock faced, with no projections of more than 3 inches from the proper line.

The belting and coping courses, as well as all quoins, shall have drafts 1½ inches wide. The coping stone shall have parallel joints dressed throughout. They shall be of such dimensions as may be required by the engineer. They shall be tied together with iron clamps made of seven-eighths of an inch square. They shall extend 9 inches within the edge of each stone, and their points shall extend 4 inches into each stone; the clamps shall be set in lead.

First-class Masonry for Piers Nos. 1, 2, 3 *and* 9.—Shall be built entirely of Ohio River freestone. The piers shall consist of headers

and stretchers, and there shall be at least one header to every three stretchers, and more frequently if necessary in the opinion of the engineer.

Headers and stretchers shall not be less than 2½ feet nor more than 6 feet long, according to thickness, and not less than 1½ feet wide, nor less in width than the depth of the courses to which they belong. In all other respects the specifications governing the construction of Piers Nos. 5, 6 and 7 shall be used in the construction of Piers Nos. 1, 2, 3 and 9.

Second-class Masonry.—Face stones shall not be less in thickness than specified for on each piece of work.

Joints.—Vertical joints on the face must be in contact at least 4 inches measured in from the face, and as much more as the stone will admit of.

Backing.—Shall be of large, well-shaped stones, having good natural or scabbled beds, the thickness corresponding to the face stones of the same course. Bond of face and backing stones shall not be less than 12 inches.

In all other respects second-class masonry shall be constructed as specified under the heads of "First-class Masonry" for Piers Nos. 5, 6 and 7.

Concrete.—Concrete shall be composed by actual measurement of four measures of broken stone of uniform size, not more than 2½ inches in any direction, free from clay and soapstone and well screened, two measures of sand and one measure of cement. The broken stone is to be well watered, and stone and mortar thoroughly turned and mixed on a tight plank floor immediately before using until every stone is coated with the mortar.

All concrete is to be laid in sections or layers, not exceeding 9 inches in thickness, and is to be thoroughly rammed. It must be mixed so dry that the water will not flush to the surface until the ramming is nearly completed. The ramming must be completed within fifteen minutes after the water has been mixed with the cement. Concrete shall be allowed at least twelve hours to set before masonry is laid on it.

Pile Foundations.—The piles shall be of white oak, not less than 8 inches in diameter at the small end and 12 inches at the butt end, with the bark peeled off. They shall be straight and carefully pointed

or shod with approved cast or wrought iron shoes, if required by the engineer.

They shall be driven to such a depth as the engineer may direct. They shall be driven with suitable rings and with a heavy hammer, with short falls, if necessary, to avoid splitting. All piles badly split or otherwise injured in driving, or driven out of place by the fault of the contractor, shall be replaced with others at the contractor's expense.

The piles shall be cut off level 2 feet below the bottom of the masonry, and 1 foot above the bottom of excavation. The pit shall then be filled with concrete to the level of the top of the piles, and the piles capped by 12 x 12-inch timbers of one length drift-bolted to the piles with iron drifts 1 inch in diameter. The spaces between the caps shall be filled with concrete, and the platform made of 12 x 12-inch timbers laid closely together.

After the first course of masonry is laid, the spaces between it and the sides of the excavation shall be filled with concrete to a height of 12 inches above the top of the platform. After the masonry has been carried above ground, the remainder of the pit shall be filled with the material excavated, well rammed in, and the pavement, if any, that was removed for the excavation, shall be carefully relaid and left in as good condition as it was before.

The spaces between the walls of ramps shall be paved, guttered and curbed with granite in accordance with the specifications in force for similar work in the City of Cincinnati.

Materials.

Stone.—The freestone shall be equal to the best quality of what is known as the Ohio River freestone, procurable on the river between Maysville and Portsmouth. The limestone shall be equal to the best quality of Indiana compact limestone, procurable near North Vernon and Greensburg, Ind.

All stone must be sound, of sufficient strength to stand the required pressure without danger of crushing, not liable to be affected by the weather, and shall be thoroughly seasoned before using.

Cement.—The cement shall be equal to the best quality of Louisville hydraulic cement. It shall stand, without breaking, a tensile strength of not less than 100 pounds per square inch in briquettes seven days old. It shall not swell or crack in the process of hardening.

Sand.—The sand shall be clean, sharp river sand, properly screened from all dirt, clay, soapstone, or other impurities.

Mortar.—The cement mortar shall be generally composed of one measure of cement to two measures sand well mixed with clear water in clear water beds and used immediately after mixing. Different proportions of sand and cement shall be used when required by the engineer.

Timber.—All the permanent timbers used in the foundations shall be of sound white oak cut from living trees, free from worm holes, dry rot, decayed and loose knots, wind shakes, and all other defects impairing its strength and durability. It shall be sawed true and of full size. Sap angles measuring over 1½ inches on the face shall not be allowed.

Iron.—All wrought iron in bolts, spikes, clamps, pile-shoes and other parts used in the substructure, shall be of the best quality of tough, ductile metal, that will stand 50 000 pounds tension per square inch before breaking, with 15 per cent. elongation in specimens three-quarters of an inch square, 12 inches long, and bend cold 180 degrees on a circle 1½ inches in diameter.

All cast iron shall be of tough, gray metal that will stand 18 000 pounds tension per square inch before breaking.

APPENDIX II.

SPECIFICATIONS FOR THE CENTRAL RAILWAY AND BRIDGE COMPANY'S HIGHWAY BRIDGE OVER THE OHIO RIVER FROM CINCINNATI, OHIO, TO NEWPORT, KY.

Form of Truss.—The form and dimensions of trusses shown on the general drawings will be satisfactory.

Strain Sheets.—The strain sheets submitted must show for each member of the truss and for cross bracing the maximum live and dead load stresses sustained, together with the wind stresses in the top and bottom chord and end brace, the dimensions and area of cross-sections, the kind of metal used, also the dead load assumed in the calculations, which must not be less than the actual weight of the structure.

Detail Drawings.—Tracings of complete detail drawings must be submitted for approval and be approved by the engineer before work is commenced. A copy of every approved strain sheet and drawing shall be furnished the engineer within ten days after its approval, and all working drawings required by the engineer will be furnished free of cost.

Material.—All parts of the structure shall be of wrought iron or steel, except washers, separating spools (over 1 inch in thickness) and ornamental work, which may be of cast iron, and the flooring hereinafter specified.

Clearance.—All through spans shall have a clear height of not less than 16 feet above the top of roadway floor, a clear width between trusses of 24 feet and two sidewalks 7 feet in the clear. The clear width between guard rails shall not be less than 22 feet.

Temperature.—Provision must be made in all parts of the structure for the expansion and contraction corresponding to a variation of 150 degrees Fahr. in temperature.

Floor.

Depth.—The depth of floor from top of roadway to lowest point of iron work shall not exceed 5 feet.

Plan.—The floor beam may be riveted to Zee iron or steel posts by means of gusset plate above pin; or, if the posts are composed of channels or built up with plates and angles, the webs of floor beams can pass through slots in the posts and be riveted to angles on the interior of same. Tension on rivet heads must be avoided.

Bottom Laterals.—The bottom laterals must be attached directly to the bottom chord pins by wing plates or by other effective means. If a stiff system is used the members must be riveted at their intersections; if rods are used they must be securely clamped together.

Roadway.—The roadway shall consist of six lines of iron stringers riveted to floor beams. These stringers shall be covered with cross timbers of suitable dimensions, spaced at not more than 30 inches centers. These cross timbers to be covered with two layers of planking, the lower course being 3 inches in thickness and not exceeding 8 inches in width, laid longitudinally with one-fourth-inch open joints. The top course will be 2½ inches thick, laid transversely and with close joints, excepting that adjacent to track rail there will be one longitud-

inal strip 8 inches in width; the remaining top course of planking must not exceed 6 inches in width. Each cross-tie must be notched over the stringers at least one-half inch, and be securely fastened to the outside flanges of outside stringers by five-eighths-inch hook bolts and in addition at two intermediate stringers, and so arranged to alternate in each consecutive tie. These hook bolts must be provided with a wrought-iron washer under nut. The cross-ties must extend without break over all stringers. The bottom course of planking shall be securely fastened to cross-ties by wrought-iron spikes at least 7 x $\frac{3}{8}$ inches, two at each end and at alternate edges over each cross-tie. The top course of planking will be securely fastened to the bottom course by fifty-penny nails of approved quality.

Guard Rail.—The wheel guards will be 8 x 8 inches, supported by blocks 3½ inches high, 10 inches wide and 10 inches long, spaced center to center a distance of 5 feet, or the distance center to center of every other cross-tie. These blocks are to be beveled at one end to a width of 8 inches at their top; the guard rail will be beveled on lower inner edge not less than 1 inch between faces of blocks; these blocks will be supported directly on the lower course of planking. The guard rail, block, lower course of planking and cross-tie shall be securely fastened by a three-quarter-inch bolt, provided with wrought-iron washers above and below.

Sidewalk.—The sidewalk floor shall consist of one course 6 x 2-inch planking laid with one-fourth-inch open joints on cross-ties of suitable size, spaced not more than 2 feet on centers, and laid transversely on iron stringers; separating strips must be used at least 6 inches long, one-fourth inch thick and 2 inches in width over each cross-tie between each line of planking. These separating strips are to be securely nailed to edge of planking. The cross-ties shall be secured to top flanges of outside stringers-by five-eighths-inch hook bolts provided with wrought-iron washers. The flooring of the sidewalk shall be fastened with forty-penny nails—two at each end of planks, and at alternate edges of each consecutive cross-tie. Suitable provision must be made for completely boxing in the lower chord of spans.

Sidewalk Railing.—A substantial iron railing, not less than 4 feet in height and of approved design, shall be erected on the outer line of sidewalks the entire length of the bridge and approaches. The railings on superstructure shall have posts resting directly on the floor

beam, and be securely braced thereto by outside braces of proper inclination, and extending nearly to the top of posts. The railings over the approaches shall be securely fastened by bolts let into the masonry and properly leaded. Intermediate stays shall be provided at center of panels.

Lighting.—Provision for lighting the entire structure must be made, using lamps and posts of design approved by the engineer.

Gas Main.—Suitable provision must be made for carrying under the sidewalk one line of 15-inch gas main.

Loads.

Dead Loads.—The structure shall be proportioned to carry the following load, viz.:

First.—The weight of iron and steel in the structure, the weight of iron being assumed at $3\frac{1}{3}$ pounds for 12 cubic inches and the weight of steel 2 per cent. heavier.

Second.—The weight of wooden floor, considering each foot B. M. to weigh $4\frac{1}{2}$ pounds for white oak. No extra allowance need be made for spikes, and railings may be assumed to weigh 30 pounds per lineal foot each. Track rails may be assumed to weigh 80 pounds per lineal foot of bridge.

The total dead panel loads will be distributed at top and bottom points as follows:

1st. On loaded chords:

(*a.*) One-half load resulting from weight of trusses.

(*b.*) The panel loads resulting from weight of lateral system in the plane of the chord.

(*c.*) One-half the weight of sway system at panel points where occurring.

(*d.*) The panel loads resulting from weight of wooden floor, floor beams, stringers, sidewalk brackets, sidewalk railings and track rails.

2d. On unloaded chords:

(*a.*) One-half the panel load resulting from weight of trusses.

(*b.*) The panel loads resulting from weight of lateral system in the plane of the chord.

(*c.*) One-half the weight of sway system at panel points where occurring.

Live Loads.—For all truss members receiving more than one panel load 75 pounds per square foot of clear roadway and sidewalks for 254-foot spans; 80 pounds per square foot for suspended span and cantilever arms; 85 pounds per square foot for 162-foot span; 100 pounds per square foot for 108-foot span. For stringers, floor beams, long suspenders and iron trestle 100 pounds per square foot of clear roadway and sidewalks, or an Aveling & Porter 15-ton steam road roller.

In the calculation of stresses the following conditions of live load will be assumed:

For main truss members, the roadway and both sidewalks will be considered loaded.

For trestle legs and long suspenders, roadway and one sidewalk only will be considered loaded, and for floor beams, the roadway will be considered loaded with sidewalks unloaded, also roadway unloaded with sidewalks loaded.

Allowed Stresses.

Allowed stresses per square inch in pounds for different members will be as follows:

Tension Members:

	Wrought Iron.	Steel.
Eye-bars and counters,	$10\,000\left(1+\frac{\text{min. stress}}{\text{max. stress}}\right)$	$12\,000\left(1+\frac{\text{min. stress}}{\text{max. stress}}\right)$
Shapes and angles (net section),	10 000	12 000
Lateral rods,	18 000	

Compression Members:

	Wrought Iron.	Steel.
Square ends,	$9\,000-30\frac{l}{r}$	$15\,000-60\frac{l}{r}$
One square and one pin end,	$9\,000-35\frac{l}{r}$	$15\,000-70\frac{l}{r}$
Pin ends,	$9\,000-40\frac{l}{r}$	$15\,000-80\frac{l}{r}$

Lateral Struts:

In which l equals distance between supports in inches, r equals least radius of gyration in inches,

$$11\,000-50\frac{l}{r}$$

Flanges of floor beam, stringers and plate girders } Tension—Same formula as for eye-bars and counters.

Compression:

In which l equals unsupported length in inches, b equals width of flange in inches,

$$\frac{10\,000}{1+\frac{l^2}{5\,000\,b^2}}$$

Alternate Tension and Compression:

For compression only. Use compression formula.

For greater stress,

Wrought Iron.

$$10\,000\left(1-\frac{\text{max. lesser stress}}{2\text{ max. greater stress}}\right)$$

Steel.

$$12\,000\left(1-\frac{\text{max. lesser stress}}{2\text{ max. greater stress}}\right)$$

Use the one giving the greater area of section.

	Wrought Iron.	Steel.
Shearing: On webs of floor beams, stringers and plate girders	4 000	
Pins and rivets	6 000	8 000
Bearing: On diameter of pin holes	12 000	18 000
On diameter of rivet holes	12 000	15 000
Bending: Stress in extreme fiber of pins	15 000	21 000
Field Rivets.—All field rivets must be iron, and provision will be made for 50 per cent. in excess of above requirements.		
Timber: On extreme fibers in bending; tension and compression	1 200	
On bearing surfaces	400	

Wind.—Wind strains shall be calculated:

1. For a pressure of 30 pounds per square foot on the exposed surfaces of both trusses and railings, and a moving load surface of 6 square feet per lineal foot of bridge.

2. For a wind pressure of 50 pounds per square foot on the exposed surfaces of both trusses and railings, the direction of wind giving larger surface being assumed in the calculations. The greatest results shall be taken in the proportioning of parts.

Lateral Struts.—Lateral struts will be proportioned to resist the resultant due to an initial stress of 10 000 pounds per square inch upon all rods attached to them, when this is in excess of wind stress. The fiber stress due to weight of strut must be considered and be deducted from the unit stress specified.

Camber.

All spans shall have an estimated camber of $d = \frac{l^2 s}{ch}$, in which d is camber in inches, l equals length of span in feet, s equals mean stress per square inch on chords in tons of 2 000 pounds, h equals depth of truss in feet, and c equals $900 + 8.4$ span for spans under 250 feet, and c equals 3 000 pounds for spans over 250 feet.

Paint.

All iron or steel, before leaving the shop, shall be cleaned from all loose scales and rust, and be given one good coat of pure linseed oil. All surfaces in contact with each other shall receive one coat of oxide of iron paint before assembling, and all planed or turned surfaces shall be coated with white lead or tallow. After erection all iron and steel work shall be thoroughly and evenly painted with two coats of paint of such quality and color as the engineer may select.

Plate Girders.

Compressed Flanges.—The compressed flanges shall be stayed transversely when their length is more than twenty-five times their width.

Web Splices.—All joints in webs shall be spliced by a plate on each side of web.

Flange Area.—No part of the web will be considered as available in flange area. The web is assumed to sustain shear only.

Stiffeners.—All web plates shall have stiffeners at the inner edges of end-bearing plates and at all points of local concentrated loadings. Intermediate stiffeners will be used if the shearing stress per square inch in web exceeds $\frac{10\,000}{1 + \frac{1}{3\,000}\left(\frac{d}{t}\right)^2}$, in which d equals clear depth between flange angles or clear distance between stiffening angles, and t equals thickness of web.

Details of Construction.

Unsymmetrical Sections.—Sections composed of two rolled or riveted channels and one plate shall have the center of pin, in all cases, in the center of gravity of the section, and any abutting members shall be so proportioned that the centers of pins shall be on the same line. All

eccentricity of stress shall be avoided. Provision must be made in all chord sections for bending from its own weight.

Bottom Laterals.—The bottom laterals will be attached directly to the bottom chord pins and not to the floor beams.

Top Lateral Struts.—The top lateral struts will have the full depth of the chord, and be securely riveted thereto.

Portals.—The portals will consist of top and bottom struts connected by cross bracing. The struts will be of neat design, be provided with ornamental brackets, and be securely attached to the trusses. The portal at each end of bridge will be provided with a name plate of approved design having such appropriate inscription as may be directed.

Top Lateral Rods.—Top lateral rods will be attached directly to the chord pins by wing plates or other effective means.

Rollers.—All spans shall have at one end nests of turned friction rollers of wrought iron or steel bearing upon planed surfaces. The rollers shall not be less than 2½ inches in diameter, and the pressure per lineal inch of roller shall not exceed 700 $\sqrt{d}$ for wrought iron or 900 $\sqrt{d}$ for steel, d representing the diameter of roller in inches.

Bed Plates.—Bed plates shall be of sufficient thickness to transmit the pressure on them uniformly to the rollers or masonry, as the case may be. It will be sufficient in determining this thickness to consider the plate having the load upon it uniformly distributed over its entire bearing surface, a continuous beam of uniform section over the walls of shoe as points of support, span lengths being taken as distances center to center of walls and from outer edge of plate to center of outer wall.

Bending moments must be taken at the center of each span and at a section cut by a plane normal to plate and parallel to walls of shoes through center of gravity of angles supporting walls to plate. The maximum bending moments must be taken. An extreme fiber strain of 18 000 pounds is allowed for steel and 15 000 pounds for iron. In no case shall the bed plates be less than 1 inch thick. The bed plates shall be so proportioned that the pressure upon masonry will not exceed 300 pounds per square inch.

Roller Plates.—Roller plates must not be less than seven-eighths of an inch thick.

Bolsters.—There will be wrought iron or steel bolsters at each end

of bridge, securely anchored to the masonry, proper provision being made for expansion.

End Braces.—End braces will have pin bearings at both ends.

Long Tension Braces.—Long intersecting tension braces shall be clamped together at intersections to avoid rattling.

Long Compression Members.—In built posts and struts of trusses the angles shall be of one length without break, but the web plates may be spliced at intermediate supports if desired.

Sub-struts and Diagonals.—Sub-struts and overhead diagonal bracing will be provided at each vertical post in through spans, when depth of truss center to center of pins is 30 feet or over.

Vertical Suspenders.—All vertical suspenders will be designed to resist compression. If eye-bars they will be stiffened by zigzag bracing or otherwise to avoid vibration.

Trestle Towers.—The base of trestle bents shall be sufficient to avoid tension under the highest wind specified and sufficient anchorage shall be provided to resist not less than one-half the overturning moment. The trestle bents shall be united in pairs to form towers and each tower thus formed shall be thoroughly braced in both directions. Cross-section and longitudinal struts shall be provided at bottom and at intermediate joints; also at top in the absence of floor beam or girders acting as such.

Raising Appliances.—At the foot of all towers and under bolsters of spans provision shall be made either by lengthening the pins, or by suitable lugs, for raising the structure to make any necessary repairs. These lugs shall be designed to resist the total weight of the structure and 1 200 pounds per lineal foot of bridge.

Effective Section of Members Built of Angles.—Whenever a member is composed of angles, both flanges of angles must be connected, else only one flange will be considered as effective.

General Conditions.

Punching.—In punching rivet holes in iron, the diameter of the punch shall not exceed the diameter of the rivet by more than $\frac{1}{16}$th of an inch, and the diameter of the die shall in no case exceed the diameter of the punch by more than $\frac{1}{16}$th of an inch.

Punching and Reaming.—All rivet holes in steel work in the canti-

lever span shall be reamed. In the other spans all holes that are not fair when parts are assembled shall be reamed at the option of the inspector. In punching steel the play between die and punch shall not be more than $\frac{1}{16}$th of an inch. In all reamed work the size of punch shall be one-eighth of an inch less than diameter of rivet to be used, and hole shall be reamed to $\frac{1}{16}$th of an inch larger than diameter of rivet. One-sixteenth of an inch must be taken out of all parts of the hole in reaming. Sharp edges of reamed holes shall be so trimmed as to make a slight fillet under the heads.

Effective Diameter of Rivets.—The effective diameter of the driven rivets in reamed holes will be assumed $\frac{1}{16}$th of an inch larger than its diameter before driving, and in making deductions for rivet holes in tension braces the same allowance will be made. In iron, this allowance must be one-eighth of an inch more than the diameter of the rivet.

Pitch of Rivets.—The pitch of rivets shall not exceed 8 inches nor be less than three diameters of the rivet. At the ends of compression members the pitch shall not exceed four diameters of the rivet for a length equal to twice the depth of the member. In stringers, where the cross-ties rest directly on the flange, the pitch of rivets must be the same throughout as at the ends.

Distance from Center of Rivet to Edge of Plate.—The distance from center of rivet to edge of plate shall not exceed eight times the thickness of plate nor be less than one and one-half times the diameter of the rivet.

Distance between Rivets in Compression Members.—The distance between centers of rivets or plates strained in compression shall not exceed twenty times the thickness of plate in line of stress nor forty times the thickness of plate at right angles to line of stress.

Length of Compression Members.—No compression member shall have a length exceeding forty-five times its least width.

Least Thickness of Plates.—No plate or shape shall be less than one-fourth of an inch thick when both faces are accessible for painting, nor less than $\frac{5}{16}$ths of an inch thick if only one face is accessible.

Bearing Plates.—All pin holes shall be reinforced, when necessary, so as not to exceed the allowed pressure on the pins, and the reinforcing plates must be provided with a sufficient number of rivets to transfer the pressure which comes upon them.

Tie Plates and Splice Plates.—The open sides of all compression members composed of two channels only, and trough-shaped sections composed of two channels and one plate, shall be stayed by tie plates at ends and diagonal lacing bars at intermediate points. The tie plates shall be square. Intermediate joints in the top chord shall be provided with tie plates at bottom, and side plates of sufficient length to hold the parts truly in position.

Lacing.—The sizes of diagonal lacing bars shall be as follows:

On the cantilever span 4 x $\frac{3}{8}$ inches.

For other spans:

$1\frac{1}{2}$ x $\frac{1}{4}$	"	"	" 7 to 8 inches.
$1\frac{3}{4}$ x $\frac{1}{4}$	"	"	" 9 to 12 "
2 x $\frac{5}{16}$	"	"	" 13 to 16 "
$2\frac{1}{4}$ x $\frac{3}{8}$	"	"	" 17 to 20 "
$2\frac{1}{2}$ x $\frac{7}{16}$	"	"	" 21 inches and upward.
$2\frac{1}{2}$ x $\frac{1}{2}$	"	"	

The distances between connections of the lacing bars shall not exceed eight times the least width of the segments connected, and in no case shall exceed an angle of 60 degrees.

Area of Rods.—No lateral or diagonal rod shall have a less area than three-quarters of a square inch.

Upset Rods.—The area at root of thread in the upset ends of rods shall be greater than the area of the rod by at least 17 per cent.

Weight of Member.—For all horizontal or inclined compression members the weight of members shall be considered, and in fixing sections the fiber stress due to weight of member shall be deducted from unit stress allowed by formula. Tensile stress shall be avoided in a transverse direction to the fiber, and shearing stress in a parallel direction to the fiber of the iron.

Effect of Wind.—If the strain from the wind in the chords or a possible temperature strain should neutralize or reverse the strain in the chord from the dead load, provision must be made for same; and if the combined strains from the dead, live and wind loads in the chords exceed 25 000 pounds per square inch, additional section must be added until the above allowed unit strain is not exceeded. Again, provision must be made in all built members for bending for wind. If the strain per square inch in such members, due to bending from wind, combined with the direct strain per square inch from dead, live

and wind loads, exceeds 25 000 pounds per square inch, additional section must be added until this allowed unit strain is not exceeded.

Washers and Nuts.—Washers and nuts shall have a uniform bearing. All nuts shall be easily accessible with a wrench for the purpose of adjustment, and shall be effectively checked after the final adjustment.

Lateral Adjustment Rods.—All lateral and adjustment rods shall be provided with open turn-buckles so that the length of thread may be verified.

Wing Plates.—The amount of metal immediately in front of pin hole in wing plate is to be determined in the following manner: The shearing area is to be considered as a section of twice the thickness of wing plate multiplied by the distance parallel to line of stress, from edge of plate to a point which is the intersection of a chord, equal to one-half the diameter of pin hole and taken normal to the line of stress, with the circumference of pin hole.

Details.—Details shall be of such nature that their strength can be accurately calculated, which strength shall be at least equal to that of the member or members which they are designed to connect.

Workmanship.

Pins and Pilots.—Pins shall be turned true to size and straight. They shall be turned down to a smaller diameter at the ends and be driven in place with a pilot nut when necessary to save the thread. There shall be a washer one-half an inch thick under each nut.

Inspection.—The inspection of work shall be made as it progresses, and at as early a period as the nature of the work permits.

All workmanship must be first-class. All abutting surfaces of compression members, except flanges of plate girders where the joints are fully spliced, must be planed or turned to even bearings so that they shall be in such contact throughout as may be obtained by such means. All finished surfaces must be protected by white lead and tallow. The rivet holes for splice plates of abutting members shall be so accurately spaced, that when the members are brought into position the hole shall be truly opposite before the rivets are driven. The chord pieces must be fitted together in the shops in lengths of at least four pieces and rivet holes in splice plates reamed while in position. Wherever it is impossible to ream together the parts which will come together in the field, the holes in both shall be reamed to an iron template.

When members are connected by bolts which transmit shearing strains, the holes must be reamed parallel, and the bolts turned to a driving fit.

Rollers must be finished perfectly round, and roller beds planed.

Rivets.—Rivets must completely fill the holes, have full heads concentric with the rivet, of a height not less than .6 diameter of the rivet, and in full contact with the surface, or be counter sunk when so required, and machine driven when practicable. Rivets must not be used in direct tension.

Built members must, when finished, be true and free from twists, kinks, buckles or open joints between the component pieces.

Eye-Bars and Pin Holes.—All pin holes must be accurately bored at right angles to the axis of the members, and in pieces not adjustable for length. No variation of more than $\frac{1}{32}$d of an inch will be allowed in the length between centers of pin holes; the diameter of the pin holes shall not exceed that of the pins by more than $\frac{1}{32}$d of an inch, nor by more than $\frac{1}{50}$th of an inch for pins under 3½ inches diameter. Eye-bars must be straight before boring; the holes must be in the center of the heads and on center line of the bars. All links belonging to the same panel, when placed in a pile, must allow the pins at each end to pass through at the same time without forcing. No welds will be allowed in the body of the eye-bars, laterals or counters, except to form loops of laterals, counters and sway rods; eyes of laterals, sway rods and counters must be bored.

The heads of eye-bars shall be so proportioned and made that the bars will preferably break in the body of the bar rather than in any part of the head or neck. The form of the head and the mode of manufacture shall be subject to the approval of the engineer. A variation from the specified dimensions of the heads of eye-bars will be allowed in thickness of $\frac{1}{32}$d of an inch and in diameter of a quarter of an inch in either direction.

Thimbles or washers must be used wherever required to fill vacant spaces on pins or bolts.

Punching and Reaming.—Rivet holes must be accurately spaced; the use of drift pins will be allowed only for bringing together the several parts forming a member, and they must not be driven with such force as to distort the metal about the holes; if the hole must be enlarged to admit the rivet, it must be reamed.

Steel Plates.—All steel plates must be straightened in the straightening machine and not by hammering.

Annealing.—In all cases where a steel piece in which the full strength is required has been partially heated, the whole piece must be subsequently annealed. All bends in steel must be made cold, or if the degree of curvature is so great as to require heating, the whole piece must be subsequently annealed.

Interpretation of Drawings and Specifications.—The decision of the engineer shall control as to the interpretation of the drawings and specifications during the execution of the work thereunder.

Distribution of Material.

All spans of the superstructure with the exception of the 108-foot span, the viaduct trestle legs and floor system in all spans shall be of steel (Class A). All rivets in steel work shall be of steel (Class B).

Quality of Material.

Wrought Iron.—All wrought iron must be tough, ductile, fibrous and of uniform quality for each class, straight, smooth, free from cinder pockets or injurious flaws, buckles, blisters or cracks.

The tensile strength, limit of elasticity and ductility shall be determined from a standard test piece not less than one-fourth of an inch thick, cut from the full-sized bar, and planed or turned parallel. The area of cross-section shall not be less than one-half square inch. The elongation shall be measured after breaking on an original length of 8 inches.

All iron shall have a limit of elasticity of not less than 26 000 pounds per square inch.

All iron used in tension shall have an ultimate strength of not less than 50 000 pounds per square inch, and elongate not less than 18 per cent.

Angles and other shapes, and plates 24 inches wide and under, shall have an ultimate strength of not less than 50 000 pounds per square inch and elongate not less than 15 per cent.

Plates over 24 inches wide shall have an ultimate strength of not less than 46 000 pounds per square inch and elongate not less than 12 per cent.

When full-size tension members are tested to prove the strength

of their connections, a reduction in their ultimate strength of (500 x width of bar) pounds per square inch will be allowed.

All iron shall bend cold 180 degrees around a curve whose diameter is twice the thickness of pieces for bar-iron and three times the thickness of pieces for plates and shapes without fracture.

Iron which is to be worked hot in the manufacture must be capable of bending sharply to a right angle at a working heat without sign of fracture.

All rivet iron must be tough and soft, and capable of bending cold until the sides are in close contact without sign of fracture on the convex side of the curve.

Steel.—Steel may be made either by the Bessemer or by the open-hearth process. All blooms, billets or slabs will be examined for surface defects, flaws and blow holes before rolling into finished sections, and such chippings and alterations must be made as will secure perfect solidity in the rolled sections.

The slabs for plates must, in all cases, be as nearly rectangular as possible and straight their whole length.

The steel must be uniform in quality for each class, and after heating to light cherry red (as seen in the dark) and quenching in cold water, shall comply with the bending requirements provided in this specification for such class of steel.

In order to grade the steel used in this work at the steel mills, the following form of selecting the test pieces shall be rigidly enforced. From every cast of metal there shall be made one test. If this test is satisfactory the whole cast may be accepted, subject to tests made on rolled sections. This test must be made from a three-quarter-inch round rolled from a 4-inch square billet, which has been reduced from original ingot; this billet to be taken during the blooming down of ingot, and reduced in such way that the reduction of section into three-quarters of an inch round will be as nearly as possible equivalent to reduction of section on finished material when rolled from original ingot. The manner and time of selecting this billet will be left to the convenience of the manufacturer.

In addition to this tension test a bending test will be required. The pieces used in this test may be either three-quarters of an inch round or three-quarters of an inch square, preferably the latter. This piece must bend cold 180 degrees about its own diameter for steel of Class A, and

180 degrees and close down flat upon itself for steel of Class B. The tests on three-quarter-inch round specimens, above mentioned, must satisfy the following requirements:

Class A.—Ultimate strength, 62 000 to 70 000 pounds per square inch. Elastic limit not less than 36 000 pounds per square inch. Elongation not less than 22 per cent. in 8 inches. Reduction at point of fracture not less than 40 per cent.

Class B.—Ultimate strength, 56 000 to 60 000. Elastic limit not less than 30 000. Elongation, 25 per cent. in 8 inches. Reduction at point of fracture, 50 per cent. Phosphorus in all steel Class A and Class B to be not over .085 of 1 per cent.

Tests at Rolling Mills.—The finished bars must be free from injurious flaws or cracks, and must have a smooth, clean finish.

At least one test will be required from every heat or furnace full of billets, slabs or blooms to prove condition of metal after rolling it into finished sections. This test piece must be cut from some rolled section of said heat and shall be generally one-half of a square inch in area and must conform to requirements as specified for three-quarter round at steel mills in every respect, except that ultimate strenght may vary 2 per cent. below minimum, and 5 per cent. above maximum.

The original number of heat or cast at steel mills must be stamped on all billets, blooms or slabs, and when rolled into finished sections this same number must be stamped on every piece rolled.

Number of Tests on Full-sized Eye-bars.—The method of making full-sized tests on eye-bars will be as follows: One full-sized test will be required in each size of bar, and if the number of bars of any size exceeds twenty, then one additional test will be required for each multiple of twenty or part thereof exceeding ten. The extra bars required for test must be ordered at the steel mills with the original order. When the Bridge Company have finished an item of bars as represented on the shop bill, the inspector shall then select from this lot of bars the bars for test, and if these tests are satisfactory the whole item may then be accepted. Should this first test fail to stand the requirements of this specification, and if in this test no blame attaches to the Bridge Company on account of poor work, the latter can demand to have two other tests made on this same item of bars, and should those two tests be satisfactory, the whole item may be accepted, and so on with other items until the whole order of bars is completed.

Heads of Eye-bars.—The manufacturers must provide sufficient excess of material in the heads of eye-bars to insure their breaking in the body rather than in the head, but any bar which breaks in the head at a stress higher than called for will be accepted; provided, only, that the elastic limit is up to specified requirements. The minimum ultimate strength of full-sized bars when tested to destruction will be 60 000 pounds per square inch. The minimum elastic limit shall be 34 000 pounds per square inch.

Castings.—All castings shall be of tough gray iron, free from injurious cold shuts or blow holes, true to pattern and of workmanlike finish. Sample pieces 1 inch square, cast from the same heat of metal in sand moulds, shall be capable of sustaining on a clear span of 4 feet 6 inches a central load of 500 pounds when tested in the rough bar.

General Requirements.

Any full-sized tension member of iron or steel tested to destruction shall be paid for at cost, less its scrap value to the contractor, if it proves satisfactory. If it does not stand the specified test, it will be considered rejected material and be solely at the cost of the contractor. The contractor shall furnish testing machine of the proper capacity, and shall prepare and test, without charge, such specimens of iron as may be required by the engineer or inspector to prove that it comes up to the requirements mentioned; he shall also furnish, prepare and test, without charge, such specimens of the several grades of steel, at steel mills and rolling mills, as may be required by the inspectors. Every facility for inspection of material and workmanship shall be furnished by the contractor, and the engineer or inspector shall be allowed full access to all parts of the establishment in which any portion of the materials are made or workmanship executed. Timely notice will be given to the engineer by the contractors when they are ready for the inspectors, and the inspectors will test and inspect the material at the mills as rapidly as it is made. All material must be inspected, weighed and stamped by the inspector before shipment. The acceptance of any material or manufactured member by the inspector shall not prevent its subsequent rejection if found defective after delivery, and such materials or members shall be replaced by the contractor without extra charge.

Engineer.—Wherever the term engineer is used throughout these

specifications, it is distinctly understood that such term shall mean the chief engineers or their authorized assistants.

Timber.—All timber must be of the best quality of white oak, sawed true to size and out of wind. It must be free from sap, except in sticks having a depth of 16 inches or upward, when 1 inch of sap will be allowed on two corners. It must be free from wind shakes, loose or rotten knots or other defects that will impair its strength or durability.

Erection.—An available portion of the river shall at all times be left open to navigation, and proper lights shall be displayed at night in accordance with the regulations and requirements of the United States Government. The contractor will furnish all staging, piling, cribbing, and material of every description required in the erection of the superstructure, and remove the same after erection is completed, leaving the river as free from obstructions as when he commenced. The contractor shall assume all risks from floods and storms, damage to persons and properties and casualties of every description until the final acceptance of the completed structure.

The erection is to be carried on subject to the approval and inspection of the engineer, and is to be completed, ready for use, to his satisfaction.

APPENDIX III.

CENTRAL RAILWAY AND BRIDGE COMPANY'S SPECIFICATIONS FOR A CAISSON FOR PIER No. 5.

The caisson to be used in building the foundation for Pier No. 5 shall be 22 x 58 feet at top, 12 feet high over all, and have a batter of one-half inch per foot from a point 1 foot below the top to the cutting edge, thus making the bottom 22 feet 11 inches by 58 feet 11 inches.

All timber used in its construction must be of a good quality of sound white or poplar, free from rotten knots, shakes or other defects, and shall be of such dimensions as shown on the plans attached hereto and forming a part of these specifications. Courses Nos. 1, 2, 10, 11 and 12 must be of oak, and as much more of the entire timber as can be secured of oak, the intention being to use no other timber except as an enforced expedient to secure the speedy execution of the work. All

framing must be accurately done so as to secure close joints throughout the work.

All the iron used in spikes, bolts, drift bolts and straps must be of a ductile, fibrous wrought iron, having a tensile strength of 50 000 pounds per square inch of section. Drift bolts must be driven into holes bored their full length, and of diameter $\frac{1}{16}$th of an inch less than the iron.

All seams between the timbers in the walls and ceiling of the air chamber, and in the outside walls, must be thoroughly caulked with oakum, and driven in with a heavy hammer. After the 3-inch sheathing is laid and thorougly spiked, the seams between the boards must also be thoroughly caulked with oakum, and wherever spike or bolt heads are to be exposed to the air pressure of the working chamber, they must be wrapped with oakum before being drawn up tight or driven home.

In order to decrease the leakage, and to secure more uniform bearing for the timbers, all the vertical seams in courses Nos. 9, 10, 11 and 12 must be poured full of a thin cement grout, mixed neat. After this is done, all the seams in course No. 12 must, in addition, be caulked with oakum.

Air lock and excavating shafts will be located as shown on the plans, and all seams in them must be thoroughly caulked. Provision must be made for carrying these shafts up through the masonry as fast as laid, either by cylinders of boiler iron or by matched oak boards capable of being made water-tight. Some approved form of iron air lock must be provided, and also excavating apparatus equal in efficiency to the "O'Conner bucket" for removing solid materials which may be encountered during the sinking.

On the sides and ends of the caisson heavy iron rings must be secured for use in suspending it during the preliminary stages of sinking, and for attaching lines to hold it in position against a swift current.

After reaching a depth satisfactory to the engineers, the air chamber and shafts must be filled with concrete made of one part by volume of cement, two parts of sand, four parts of limestone, rock broken so as to pass through a ring $2\frac{1}{2}$ inches in diameter, and sufficient water to bring the mortar to proper consistency, the whole to be thoroughly rammed before the setting of the cement.

The triangular space between the air chamber and outside walls must also be filled with concrete, mixed in the proportions above stated, and as the superimposed course of timber is being laid, care must be taken that each stick is well bedded in a freshly mixed mortar spread on top of the concrete.